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DEEPWATER PILOT PLANT TREATABILITY STUDY

Gilbert Horwitz

Delaware River Basin Commission
Trenton, New Jersey

March 1974

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16. Abstract

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Aerobic digestion reduced excess activated sludge volatile suspended solids 50 percent in 20 days. Either vacuum filtration or filter pressing would be most applicable for dewatering.

Pilot plant operation confirmed treatability proposals, developed design criteria and pointed out areas of ~~concern for~~ additional study. (EPA)

17a. Descriptors

*Pilot Plants, *Activated Sludge, *Regional Analysis, Aerobic digestion, Filtration, Activated Carbon, Organic Loading, *Delaware River Basin Commission, *New Jersey, *Waste water treatment, Pre-treatment (water), Neutralization

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March 1974

Final Report

DEEPWATER PILOT PLANT TREATABILITY STUDY

by

Delaware River Basin Commission
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The Delaware River Basin Commission initiated a study of a joint industrial-municipal regional wastewater collection and treatment system for southern New Jersey. Staff personnel determined an optimum collection area for ten industrial plants and inclusive municipalities.

Engineering-Science, Inc. was selected as design and operating engineers of a 50 gpm pilot plant to treat a composite of refinery, petrochemical, and municipal wastewater.

Raw wastewater was subjected to the following processes: pretreatment, equalization, neutralization, primary clarification, varied types of activated sludge, final clarification, and intermittent varied testing on polishing and disinfection.

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Aerobic digestion reduced excess activated sludge volatile suspended solids 50 percent in 20 days. Either vacuum filtration or filter pressing would be most applicable for dewatering.

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This report was submitted in fulfillment of Project Number 11060-DR0 under the sponsorship of the Environmental Protection Agency.

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SECTION I

PREFACE

Onset of Regionalization

The DRBC established Standards for water quality control in April 1967. After adoption of the Standards, there was considerable local interest in regionalization to minimize waste treatment costs, particularly by industry. This reaction reinforced the Commission Director's feeling that serious and detailed thought should be given to regionalization, particularly along the Delaware Estuary.

Pollution Abatement for the Delaware Estuary

The most critical water quality problem area in the Delaware River Basin is the 86 mile long Estuary stretching from Trenton, New Jersey to Liston Point, Delaware. The Estuary receives waste discharges from a complex, broad spectrum of industry and several major cities: Philadelphia, Pennsylvania; Trenton, New Jersey; Wilmington, Delaware; Camden, New Jersey; and Chester, Pennsylvania.

To meet or exceed the dissolved oxygen concentration set by the Standards and also sustain the other uses specified by DRBC, the capacity of the Estuary to accept waste was allocated among the dischargers. Based on the Year 1964 raw wastewater data, treatment reductions of approximately 88% of the first stage oxygen demand are required. With the growth anticipated in this area, higher treatment reductions may be necessary by Year 1990.

Initial Development of Deepwater Regional Study

The Deepwater area extending some 30 miles in Gloucester and Salem Counties in southern New Jersey appeared to be economically and practically favorable for waste treatment regionalization.

An abbreviated preliminary evaluation of the technical feasibility of a two-county regional waste treatment facility in the vicinity of Deepwater, New Jersey was conducted by DRBC staff in January 1968. The determination of the boundary of the collection system for regionalization was based on balancing the cost advantages of a regional treatment facility against the cost of interceptors to convey wastes to the central location. Preliminary indications showed that the optimum 130 MGD collection system would extend from the City of Salem, Salem County to Mobil Oil Corporation, Gloucester County with the regional treatment at Deepwater, New Jersey adjacent to the DuPont-Chambers Works plant which had the bulk of the flow. Subsequent development showed that inclusion of Texaco, Inc. further upriver from Mobil would serve the purpose of covering all major riverfront industries in Gloucester County; the significant industries in Salem County were already included. In addition, municipal wastes within the collection area of the two counties would be included in the regional system. The system would extend from the regional treatment facility 23 miles upriver to Texaco and 9 miles downriver to the City of Salem. Ten industrial plants and four municipalities were considered for the collection area. This area is shown on the following map.

Mathematical model evaluation conducted by DRBC showed an overall improvement in the dissolved oxygen profile by translating the wastes to the proposed regional plant and outfall. There appeared to be about a 10% cost advantage for the regional system based on total annual costs for initial development considering amortization and operation and maintenance.

Meetings were held with the industries and municipalities and these showed favorable support to continue the study.

Study Proposals

It was decided that an in-depth study by a consulting firm would be required to delineate the collection area, determine a preliminary engineering cost estimate of the project and develop these details necessary to determine treatability. The project was divided into two basic studies: a pilot plant study to determine treatability and a traditional preliminary engineering study that would be developed concurrently with the pilot plant study. It was envisioned that the pilot plant study would eventually have a feed-back into the costs pertaining to the preliminary engineering for the treatment facility. This report encompasses the pilot plant operations.

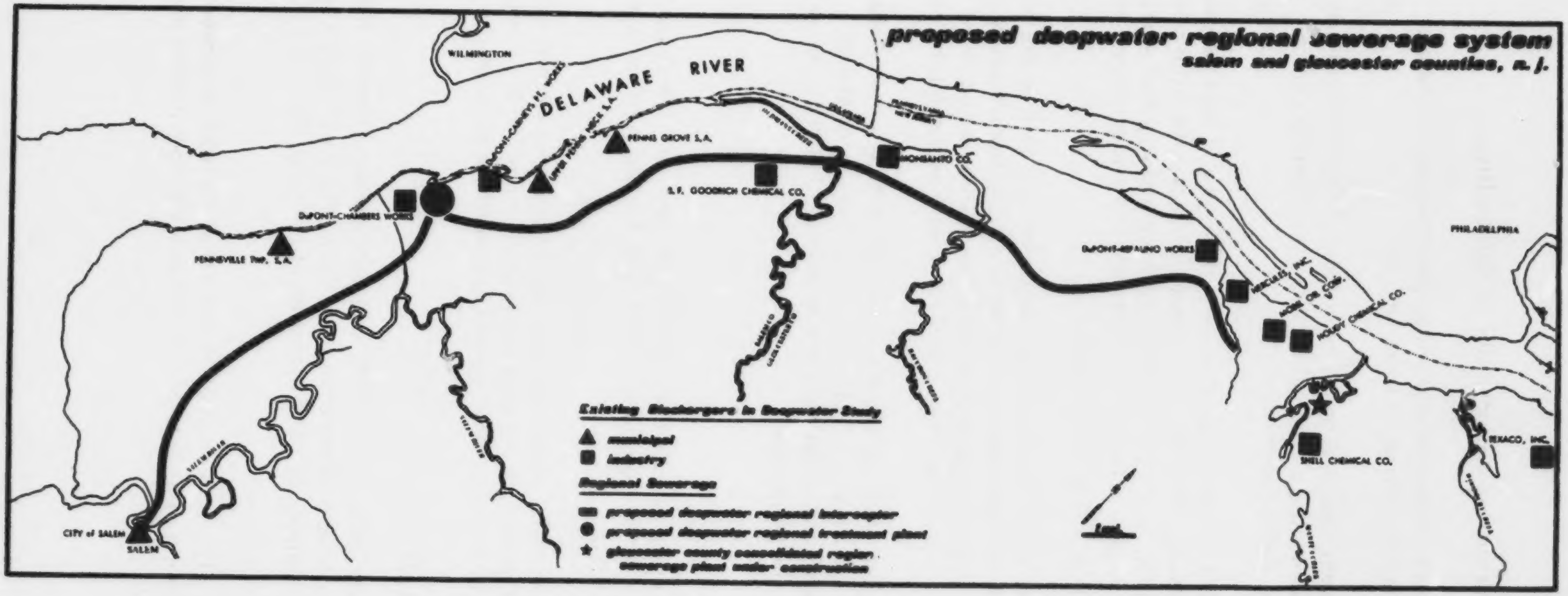
Pilot Plant Study

The specific objectives of the pilot plant study were:

a. to determine the treatability characteristics of the composite industrial and municipal wastes;

b. to develop design criteria for the facility to achieve 90-95% BOD reduction as well as to meet other effluent quality requirements;

c. to test methods of secondary and advanced waste treatment of combined municipal and industrial wastes;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salem and gloucester counties, n. j.
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Existing Dischargers in Deepwater Study
▲ municipal
■ industry
Regional Sewerage
proposed deepwater regional interceptor
● proposed deepwater regional treatment plant
★ gloucester county consolidated region sewerage plant under construction

Figure P-1

d. to estimate cost of construction and operation of the facility;

e. to provide data on which to base an equitable apportionment of the cost among the industries and municipalities to be served; and

f. to demonstrate the expedituous and timely resolution of the technical and economic difficulties of achieving a regional solution to a complex multi-industrial and multi-waste disposal problem.

The pilot plant was planned to operate continuously at 50 gpm, 7 days per week, 24 hours per day. Wastewater was to be composited with projected loads and flows so as to be representative of the influent to the proposed regional waste treatment plant. Of the 50 gpm entering the pilot plant, it was estimated 44 gpm to be industrial wastewater and 6 gpm municipal.

Wastes from the nearby DuPont-Chambers Works would be conveyed to the pilot plant by ditch flow. Tank trailers were envisioned to convey the composite waste from each industry. It was envisioned that several tank trailers would bring in the waste from varied distances between 2 and 25 miles from the pilot plant.

Treatment processes proposed were equalization, neutralization, primary clarification, aeration, final clarification, polishing, chlorination and sludge disposal. The aeration basin was to be designed to provide flexibility for various methods of the activated sludge process.

Funding

Federal funding assistance was solicited and an EPA Research and Development grant offer of $646,700 or 67% of eligible project costs for the pilot study was received on March 24, 1969.

On May 21, 1969, ten industrial plants and four municipalities agreed to contribute up to $654,300 to fund a portion of the pilot plant study and all of the total preliminary engineering study. The DRBC agreed to contribute $75,000.

Contract for Engineering Services

A contact was entered into between the DRBC and Engineering-Science, Inc., on June 27, 1969 for the major part of the studies including the preliminary engineering studies and the design, construction, and operation of the pilot plant and evaluation of data.

The timetable included: (1) completion of an interim preliminary engineering report not later than 6 months after date of the contract; (2) construction and operation of the pilot plant not later than 6 months after date of the contract; (3) completion of the final report on the preliminary engineering studies not later than one year from date of contract; (4) completion of the interim report on pilot plant treatment studies not later than 18 months after date of contract; and (5) completion of a final report of pilot plant studies not later than 3 years from date of the contract. Scheduled dates were met.

Project Status at Completion of Pilot Plant Study

At the completion of the study, the industries were unwilling to agree to participate in full scale regional treatment without guarantees of state and federal construction grants. Such guarantees could not be met which was a major cause of the full scale regional system not developing. The basis for industrial resistance was that approximately one-quarter of the construction cost of the initial project was the incremental cost for system capacity for future participants. Industry did not want to subsidize facilities for other, possibly competitive, industry without a guarantee of public aid. However, one of the alternate plans of regional treatment with the dischargers split into two regional systems - upriver and downriver portions - is presently being pursued at county level.

The data gathered as the result of the pilot plant operations have been utilized by several of the participating entities. The study engendered considerable interest, nationwide, and a number of requests have been received for copies of the report. This project provided an example of a solution to a difficult problem that could be applied nationwide.

CONCLUSIONS AND RECOMMENDATIONS

An analytical and treatability program for an area-wide industrial and municipal treatment facility has been consummated and found to be sufficiently complete to develop design information. The bench and pilot scale testing programs, covering a period of 30 months, were established to obtain the design coefficients and parameters. On this basis, cost estimates were formulated for a proposed regional treatment system which could serve industries and municipalities in the lower Delaware River Basin.

CONCLUSIONS

An analyses of data accumulated during the course of this project and presented in the report has resulted in the following conclusions and observations.

Wastewater Characterization

1. The wastewaters from each of the participating industries were characterized with respect to their organic and inorganic quality. This characterization schedule was implemented during both the bench scale and pilot plant phases of this project. The analytical results indicate that the combined wastewater conveyed to the regional treatment system will have the following general characteristics:

Chemical Oxygen Demand (COD)	420-822 mg/l
Biochemical Oxygen Demand (BOD)	136-453 mg/l
Total Organic Carbon (TOC)	109-358 mg/l
Phenols	1-19 mg/l
Phosphates	0.2-13 mg/l
Total Kjeldahl Nitrogen	10.5-45 mg/l
Color	200-1800 Std. Co. Units

2. It is recognized that the aforementioned concentrations of organic constituents as well as inorganic levels will fluctuate with both seasonal and operational variations. Although one must recognize the factors which are prevalent when interpreting these data--namely, sampling methods and frequencies, analytical procedures, interferences, etc.--it does provide a rational approach for establishing an individual and collective characterization picture of the combined wastewater which must be treated in the regional system. It is from this information that plant design, cost evaluations, and cost allocations must be formulated.

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Bench Scale Treatability Studies

1. Based on studies and considerations established during the early phases of this project, the need for an equalization basin at the site of the regional wastewater treatment plant does not appear to be economically attractive or economically justified.

2. The results of neutralization studies conducted during this phase of the project indicate that approximately 8.7 meq/l of lime are required for neutralization. The most acidic conditions encountered during the analytical program required approximately twice as much lime as normally required. As unusual operating conditions would have to appear simultaneously at several of the participating plants, it is doubtful that the pH of the unneutralized stream would ever be above 7.0. The need for acid neutralization is therefore not envisioned.

3. The results of the coagulation-precipitation studies indicated that chemical precipitation of the combined wastewater flow at the regional plant as a method of removing organic constituents is not justified.

4. It is estimated that 95 percent of the BOD_5 contained in the combined wastewaters can be removed at a loading of 0.25 lbs BOD_5/day/lb MLVSS with approximately a 90 percent BOD_5 removal at loadings up to 0.70 lbs BOD_5/day/lb MLVSS. These data were obtained using bench scale biological reactors.

5. It is anticipated that approximately 36,000 lbs/day of biological synthesis sludge and 144,000 lbs/day of primary sludge will be generated when the full-scale treatment system is put into operation.

6. The bench scale studies indicate that approximately 1,800 lbs of oxygen/day/MG are required for an aerobic activated sludge system.

7. Based on laboratory analyses, fecal organisms observed in the raw industrial wastewaters appeared to be sufficiently destroyed as to not require disinfection. These studies indicate chlorination of the final effluent from a treatment system receiving wastewaters as presently constituted is not required. Coliform analyses conducted during the pilot plant phase of this study confirmed these results.

8. The effectiveness of removing pollutants from biologically treated effluent using carbon adsorption was evaluated by undertaking batch carbon adsorption isotherm tests. These studies indicated that most of the soluble BOD remaining after biological treatment was removed with an activated carbon dose of less than 40 mg/l. The removal of color to trace levels required carbon concentrations slightly in excess of 200 mg/l. While batch isotherm studies are "screening tests" only, they are indicative of carbon removal capacities and thus establish

a basis for subsequent continuous column studies.

Pilot Plant Treatability Studies

1. The hydraulic mixing characteristics for the 50 gpm capacity pilot plant facility were established using dye studies. The results for the equalization tank indicate that adequate mixing and circulation are achieved by using a high capacity recycle pump. The data indicate that only 23 percent of the tank was unused, with the remaining volume exhibiting completely mixed characteristics. Neutralization and aeration tanks were completely mixed. The primary and secondary clarifier flow patterns were found to be adequate after subsequent modifications were made.

2. The oxygenation capacity of the mechanical and diffused aeration systems in the aeration tanks of the pilot plant were determined. A transfer efficiency of 2.90 lbs of oxygen/HP-hr for the mechanical aeration system was noted and an efficiency of 1.15 lbs of oxygen/HP-hr was observed for the diffused aeration system. These results are reflected in the process design formulation.

3. A computer program was developed to perform the necessary mathematical operations of the biological treatment results and resolve the data into design parameters and coefficients with an interpretation of the statistical reliability of each parameter.

4. The biological pilot plant studies indicate that approximately 65 percent of the COD and 90 percent of the BOD_5 can be effectively removed by this process except during periods of extremely cold weather. A minimum BOD_5 removal of 66 percent is predicted during the coldest period of the year based on a temperature balance calculated across the aeration basins. As this balance was made utilizing observed inlet temperatures to the aeration basin of the pilot plant, slightly higher removal efficiencies could be expected in the full scale plant based on a comparison of heat losses calculated for both the pilot plant and the regional facility.

5. The transient loading studies conducted during both the summer and winter operations indicate that there is little or no effect on the performance of the biological system due to variations in the organic characteristics of the combined wastewaters. Equalization at the regional treatment site, therefore, is not recommended based on these results.

6. The process design criteria for the biological system were formulated based on the computer resolved design parameters and coefficients. These basic criteria include a required aeration detention time of 12 hours, an oxygen utilization rate of approximately 155,000 lbs/day and a biological sludge production rate of

36,000 lbs/day with a hydraulic design flow rate of 72 MGD.

7. The pilot plant performed efficiently in the removal of organic contaminants from the combined wastewaters. However, there were occasional problems encountered during the operations which should be considered in the full-scale treatment design. For example, the pH monitoring probes required cleaning; foaming occurred occasionally in the aeration basins and was excessive at times; exposed carbon steel appurtenances on the upstream side of the neutralization facility were subject to extensive corrosion; and occasional power shutdowns temporarily interrupted operations. The experience gained from the pilot plant operations is considered invaluable in the translation of these studies to full-scale design and implementation of wastewater management practices.

8. Aerobic digestion provides a maximum of 50 percent reduction in volatile suspended solids (VSS) with a retention period of 20 days. A retention time of seven days is sufficient to achieve 75 percent reduction of the digestible solids provided the reactor has facilities for continuous withdrawal of sludge liquor and subsequent thickening of the residual sludges. Mixing will control requirements for an aerobic digester.

9. The effectiveness of the filter press, vacuum filter and centrifuge for dewatering primary and digested sludges was evaluated using pilot scale models. These process simulation studies indicate that filter press or vacuum filtration dewatering would be the most applicable processes when considering land fill as the ultimate disposal of the sludges. Filter press dewatering of the combined primary and excess biological sludges was used for the process design calculation and cost estimates.

10. Continuous flow bench and pilot scale carbon column studies indicate that carbon adsorption is effective in removing color, residual organics, and toxic substances from the biologically treated effluent throughout the operational year. However, the data suggest that carbon adsorption is more effective as a tertiary process following biological treatment than as a total process. This observation is predicated on the fact that a high leakage of BOD was noted when the untreated wastewater was applied directly to the carbon columns. This can be attributed to the presence of certain organic constituents which are biodegradable but not amenable to adsorption.

11. The addition of powdered activated carbon to the activated sludge process was investigated. Although improved organic removal was observed, the sludge handling phase including powdered carbon regeneration has not been sufficiently developed to allow a forceful recommendation of this process.

12. Effluent polishing processes including sand filtration and microstraining were investigated. On the basis of effluent quality criteria, the use of either of these processes as polishing processes is not recommended.

Effluent Quality Analysis

1. The predicted effluent quality of the proposed treatment facility as presented in this report will meet the DRBC standards as adopted on 7 March 1968, and as amended through March 26, 1970.

2. It is recognized that the effluent quality projection as presented in this report is based on the treatability of the combined wastewater having the quality characteristics observed during this study. However, the period of time over which the treated and untreated wastewaters were characterized affords statistical creditability. The effluent quality as predicted is therefore sufficiently accurate to justify implementation of the recommended system which has the capacity and capability to treat wastewaters of a similar nature to this required quality level.

RECOMMENDATIONS

Based on the conclusions as stated herein, and the detailed investigations which are documented in this report, the following recommendations are made concerning the major treatment processes for the regional treatment system.

1. It is recommended that the major treatment processes for the regional treatment system, based on economic considerations, process applicability and effluent quality standards, include an activated sludge system followed by an activated carbon effluent polishing system. Recommended pretreatment processes include neutralization and primary clarification. Additionally, aerobic digestion and filter press dewatering are recommended to handle the primary and wasted activated sludges.

Specific recommendations pertaining to the individual treatment processes are as follows:

a. It is recommended that the neutralization system includes a premixing basin prior to a series of four, two-stage neutralization basins, each stage having a residence time of fifteen minutes. Dolomitic lime is recommended as the neutralization agent based upon economic considerations.

b. It is recommended that the primary clarification system includes twelve parallel basins equipped with mechanical sludge removal mechanisms. Each basin would be sized for a maximum overflow rate of 800 gal/day/

ft^2 with a minimum residence time of two hours.

c. It is recommended that the secondary biological system include six completely mixed, parallel aeration basins each having a residence time of twelve hours. Ten - 100 HP, pier mounted, surface aerators are recommended for each basin to provide for adequate mixing and oxygenation. Final clarification will be accomplished by twelve, parallel, center-fed, 110 ft. diameter circular clarifiers.

d. It is recommended that the activated carbon effluent polishing system include twenty, two stage parallel adsorbers having a total residence time of 20 minutes. Activated carbon regeneration, storage and conveyance appurtenances should be sized to handle a carbon exhaustion rate of 283,300 lbs/day.

e. It is recommended that the wasted activated sludge be aerobically digested, combined with the primary sludge, gravity thickened and then dewatered by the filter press process. Ultimate sludge disposal by land fill is recommended as this is currently the most acceptable method. (Reference Interim Pilot Plant Report, Chapter VII).

2. It is recommended that a 72 MGD treatment facility, conceptually designed as described within this text, be implemented to serve the industries and municipalities in the lower Delaware River Basin. The estimated capital cost of the regional facility is $39,957,000(ENR=1400). The estimated annual operation and maintenance cost is $2,965,000 and the total annual cost is estimated to be $5,829,000. While it is recognized that a higher ENR value would be applicable, 1400 is used to be consistent with previously submitted cost estimates.

SECTION III

INTRODUCTION

GENERAL

This final pilot plant report summarizes the results of the entire pilot plant testing program. This compilation is intended to complement the final report, "Deepwater Regional Sewerage System Preliminary Engineering and Feasibility Study," presented by Engineering-Science, Inc., to the Delaware River Basin Commission, June, 1970. This project was supported in part by the United States Government, Environmental Protection Agency, Research and Development Grant 11060-DR0. Where that report presented the aspects related to the Interceptor Preliminary Engineering Study system cost estimates and cost apportionment plans, the information presented herein relates to the "treatment phase" of the overall project. These data represent the accumulation of approximately 30 months of treatment studies, including the purification of wastewaters emanating from the participating industries and municipalities, the handling of resultant wastewater sludges, and an evaluation of applicable wastewater treatment and handling systems, using both bench scale and pilot scale unit procedural techniques.

The use of a combination of unit processes, which must be properly integrated in order to constitute an efficient waste treatment system, depends on many factors. A "treatability" evaluation, therefore, must consider and properly define these factors in order to effectively translate the data as presented into a basis for establishing an optimal treatment system. The presentation of treatment information, its interpretation, and its resolution to design information is therefore consistent with the goals of conceptualizing and developing a wastewater treatment complex capable of producing an effluent with a quality which meets the criteria as established by the regulatory authority.

SCOPE OF THE STUDY

The scope of the Deepwater Pilot Plant Study generally conforms with that outlined in the "P Task" section of the proposal for the Deepwater Pilot Plant Engineering and Interceptor Preliminary Engineering Study submitted to the Delaware River Basin Commission by Engineering-Science, Inc., in February, 1969, and incorporated into the contract between the aforementioned parties in July, 1969. However, there are many ancillary studies both with respect to treatment of liquid wastewaters and handling of sludges, which were not included in the original scope but which were considered necessary in order to fully complement the treatment evaluation program.

As defined in the original proposal presented by Engineering-Science, Inc., the project was subdivided into a series of individual and identifiable Tasks. The identification of these Tasks, properly sequenced, is illustrated in the Activity Plan as originally presented, Figure 1. The implementation of these various tasks generally follows the format originally established. This Task delineation of project requirements can be summarized as follows:

Task P-1 -- Preliminary Design of Pilot Plant Characteristics

This Task includes a comprehensive wastewater collection and characterization program, bench scale biological treatability studies both on the individual wastewaters and the composite, an evaluation of the related physical and chemical characteristscs of the biological system, and ancillary bench scale studies necessary for overall system evaluation.

Task P-2 -- Define Pilot Treatment Plant Programs

This Task involves the comprehensive review of Task P-1, as well as other inputs, all related to properly defining the pilot treatment plant program. Alternative pilot treatment systems were considered, methods for properly collecting and analyzing data were delineated and operational flexibility requirements were defined.

Task P-3 -- Design Pilot Treatment Plant

Based on preliminary information collected in Task P-1 and elsewhere, the final pilot treatment plant design drawings and specifications were formulated.

Task P-4 -- Construct Pilot Treatment Plant

Once the design drawings and specifications were reviewed and approved, the Deepwater Pilot Plant, which was designed hydraulically at 50 gallons per minute, was constructed. This construction included wastewater receiving facilities, storage tanks, neutralization, biological oxidation processes, final clarification, and chemical treatment facilities. All of the piping, control valves, sample collection devices, and process safeguard appurtenances were included in order to insure that the pilot system would be capable of meeting project objectives.

Task P-5 -- Evaluate Unit Processes of Pilot Treatment Plant

The physical, hydraulic and oxygenation characteristics of the pilot plant facility were established in this Task effort.

Figure 1

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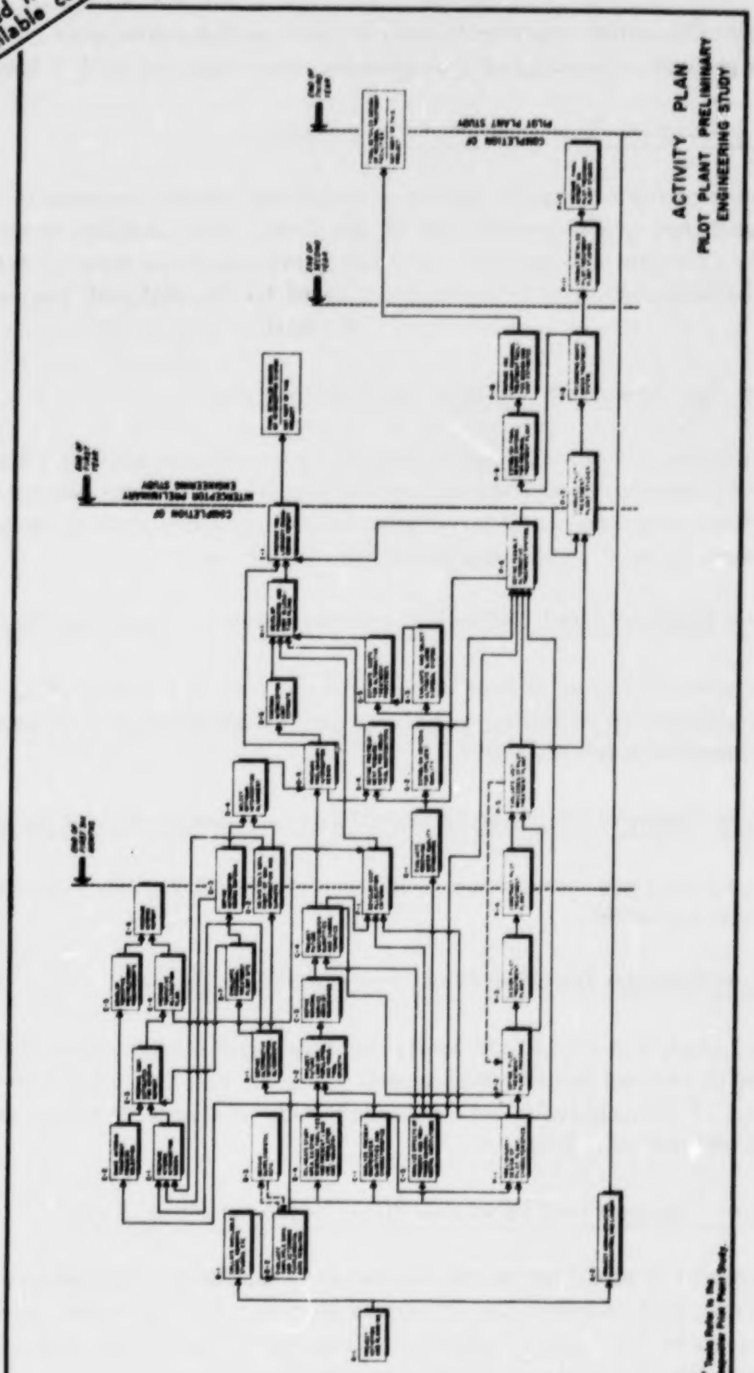
ACTIVITY PLAN
PILOT PLANT PRELIMINARY ENGINEERING STUDY
COMPLETION OF PILOT PLANT STUDY
COMPLETION OF INTERCEPTOR PRELIMINARY ENGINEERING STUDY
Note "P" Tasks Refer to the Deepwater Pilot Plant Study.

Task P-6 -- Define Feasible Alternative Treatment Systems

The feasible alternative treatment systems based on the wastewater characterization per present technology and economics were assessed in this Task write-up.

Task P-7 -- Conduct Pilot Treatment Plant Studies

The responses of the biological system to operating and environmental variables were assessed during the performance of this Task. Additionally, ancillary biological, chemical and physical tests were undertaken in order to establish a basis for the formulation of final design criteria for the regional treatment facility.

Task P-8 -- Recommend Wastewater Treatment System

Additional modes of treatment were considered in order to establish the relative feasibility of using conventional biological processes. Carbon adsorption and chemical treatment were considered both in terms of individual systems or as supplementary steps to the biological phase of treatment.

Task P-9 -- Establish Final Design Criteria for Regional Treatment Plant

Based on a comprehensive review of the "P Tasks" up to and including P-8, general guidelines for selecting and sizing unit processes within an overall treatment complex were set forth.

Task P-10 -- Prepare Preliminary Regional Facility Design & Cost Estimates

This Task included the formulation of the general treatment plant design and the resultant cost estimates.

Task P-11 -- Conduct Detailed Pilot Treatment Plant Studies

Following completion of the previously outlined pilot studies, additional tests considering a refined treatment approach using the existing pilot system were undertaken. This included a more thorough study of sludge handling and disposal and liquid effluent polishing.

Task P-12 -- Prepare Final Report on Pilot Treatment Plant Study

This final report is submitted to the Delaware River Basin Commission and includes the entire spectrum of previously discussed studies. The submission of this final report constitutes the terminal phase of the project, the timing of which is in accordance with that outlined in the Activity Plan, Figure I.

ORGANIZATION OF THE STUDY

In order to effectively implement the Tasks associated with the bench and pilot scale treatability program, key personnel including engineers, chemists, and management specialists were selected. This team then directed their entire efforts toward the realization of the project objectives.

A special office was established by Engineering-Science, Inc. at the pilot plant site located on the Chambers Works Plant, E. I. duPont de Nemours Company, Deepwater, New Jersey. The field supervision of the wastewater collection program, bench scale treatability studies, pilot plant construction and operations, and data assimilation and processing was conducted from this office.

Overall project management for the wastewater treatment phase of this project was provided by Eugene J. Kazmierczak, President of Engineering-Science, Inc., and Dr. Harvey F. Ludwig, Chairman of the Board. Dr. Davis L. Ford was the Project Engineer assisted by Resident Engineers Fred J. Fahlen and S. Dave Ellison. Staff engineers who provided valuable assistance to this project include Dr. Jan Scherfig, Nicholas L. Presecan, Larry Tropea, James M. Eller, Billy A. Carnes, Richard W. Bentwood, and Douglas M. Darden.

The analytical work associated with this project, including organic analyses, bacteriological testing, and ancillary chemical, physical, and biological analyses were conducted by contractual arrangement between Engineering-Science, Inc., and duPont. Trucking of the various wastewaters to the pilot plant holding tanks was undertaken by Chemical Leaman, Inc., and the daily operational and maintenance duties were relegated to duPont personnel, all according to agreements with and under the supervision of Engineering-Science project management.

Special Consultants to Engineering-Science, Inc.

The consultants which provided special input to the design and implementation of the pilot plant program are eminently qualified in the field of wastewater treatment and water quality management. They are:

(1) Dr. Earnest F. Gloyna, Dean, College of Engineering, The University of Texas at Austin, Austin, Texas, and;
(2) Dr. Erman A. Pearson, Professor of Sanitary Engineering, University of California at Berkeley, Berkeley, California.

Coordination and Liason

Dr. Leon Weinberger, former Assistant Commissioner of the Federal Water Quality Administration, and Dr. Gordon McCallum, former Assistant Surgeon General,

U. S. Public Health Service, both of the Washington, D. C. office of Engineering-Science, served in the capacity of providing the necessary liason with the Environmental Protection Agency in Washington, D. C.

Delaware River Basin Commission Staff

The project implementation was continuously coordinated with the Delaware River Basin Commission Staff including the following: James F. Wright, Executive Director; Herbert A. Howlett, Chief Engineer; Ralph Porges, Head, Water Quality Branch; Paul Webber, Supervisor of the Deepwater Project; and Arthur E. Peeck, Chief Administrative Officer. This coordination was necessary in order to insure that Task development and implementation were commensurate with the general project goals and water quality objectives established by the Commission.

Technical Advisory Committee

Each participating industry and municipality had representation on the Technical Advisory Committee and this consortium provided valuable assistance and guidance throughout the conduct of the Project. Mr. W. H. Roach of Texaco, Inc., served capably as Chairman of this committee until his retirement from Texaco. Mr. Charles A. Evans of the duPont Chambers Works Plant succeeded him as Chairman. Mr. Robert Kausch has ably served as secretary of all Technical Advisory Committee meetings. Monthly meetings were held by the TAC in order to provide a forum for submitting progress reports, exchanging ideas, and insuring liason between all of the attendant groups.

Executive Committee

Mr. Herbert A. Howlett, Chief Engineer of the Delaware River Basin Commission, and Dr. Harvey F. Ludwig, Chairman of the Board, Engineering-Science, Inc., assisted by their staff consultants, have reviewed the treatability phase of the project, and in concert with Mr. James F. Wright, have made recommendations relative to its effective implementation.

Acknowledgements

There are many entities and individuals who have made significant contributions to the technical and managerial facets of this Project. Those organizations and individuals previously mentioned deserve special credit as well as the Environmental Protection Agency, the Department of Environmental Protection of the State of New Jersey, and the Delaware River Port Authority. Particular appreciation is expressed to industrial and municipal representatives who assisted Engineering-Science personnel in resolving the complex logistics involved with collecting representative wastewater samples from the many points of discharge within the study area.

SECTION IV

WASTEWATER CHARACTERIZATION

The first step in developing a rational basis for designing wastewater treatment facilities is the determination of the wastewater characteristics, both quantitatively and qualitatively. This is particularly complex when considering the variable flow and constituents inherent with the operations of both the participating municipalities and industries. Because of this variation, it is necessary to obtain sufficient characterization data to have statistical significance. Moreover, it is necessary not only to define existing quantities of pollutants but also to project pollutant levels which could be anticipated throughout the life of the treatment facility.

The present and projected industrial wastewater quantities established for purposes of designing a regional treatment facility in the Deepwater Region have been cited previously. However, a more complete tabulation of wastewater characterization data collected during the bench scale and pilot scale phases of this Project are presented herein.

The characterization schedule included those parameters considered meaningful with respect to wastewater definition, treatability, and effluent quality requirements. Because of the volume of data accumulated during the course of this study, only pertinent statistical results are reported in this Chapter. Additional information - such as sample collection procedures, data correlation and interpretation, and ancillary parametric definition - is also included.

DESCRIPTION OF THE SAMPLING PROGRAM

Sampling programs were established at all the industrial sites in the Study Area for the bench scale phase of the study, with the exception of the B. F. Goodrich Plant which was under construction. Prior to the initiation of the sampling programs, in-plant surveys were made at each industry to determine the layout of the wastewater systems and to select sampling points where the most representative samples could be obtained. Sampling schedules were then initiated at each individual plant depending on the type of sampling equipment utilized and the sample frequency required to obtain the composites. Each composite was then collected for analysis and transported to the laboratory.

Sampling programs at five municipal treatment plants in the Study Area were established on a 24-hour composite basis with three composites taken on Tuesday, Thursday, and Saturday. Municipal treatment plants sampled included Pennsville Sewerage System, Salem City, Upper Penns Neck, Woodbury, and Paulsboro.

A description of the wastewater facilities at each of the industries, the sampling sites, and the sampling programs that were established are described below:

Industry: Texaco, Inc.

Treatment Facilities

The treatment facilities at Texaco consist of a collection system that discharges into an API oil separator. The effluent from the separator flows to a surface discharge point, over a five foot rectangular weir, and into the Delaware River. All process wastewater, cooling tower blowdown, boiler blowdown, process area runoff and ballast water from incoming ships flow via the collection system to the oil separator.

Sampling Site and Equipment

The economics of oil recovery through the separator dictates oil separation prior to the discharge of the wastewater. Therefore, the proposed discharge to the regional system would be the effluent from the oil separator. A sampling point was established at the separator outfall.

The sampling equipment consisted of a gas-operated liquid sampler (Protec Model). This instrument was set so that a series of 50 ml samples, taken at specific time intervals, would give a sample volume of 22 liters over each 24-hour composite period. This type of compositing is considered satisfactory because the flow from the continuous refining process is relatively constant.

Sampling Program

Twenty-four hour composites were taken every other 24-hour period so that within a two-week sampling period, each day of the week was represented.

Industry: Shell Chemical Co.

Treatment Facilities

The treatment facilities at Shell consist of a neutralization chamber, floatable solids separation tank and a lift station-force main system that delivers the effluent to the Delaware River. There are three separate collection systems within the plant. Two systems flow directly to the neutralization chamber, one conveying the septic tank overflow, cooling tower blowdown and some process wastes and the other conveying the effluent from the alcohol recovery unit. The third system is the surface runoff collection sewer which empites into Mantua Creek without treatment.

Sampling Site and Equipment

The effluent from the process area contains floatable plastic fines and therefore, the proposed effluent to the regional system would be the effluent from the floatable solids separation tank. The sampling point was established at the outfall of the solids separation tank.

The sampling equipment consisted of a Protec Model, gas-operated, liquid sampler. This instrument was set so that a series of 50 ml samples, taken at specific time intervals, would give a sample volume of 22 liters over each 24-hour composite period. This type of compositing was considered satisfactory because the flow from the polypropylene process is relatively constant.

Sampling Program

Twenty-four hour composites were taken every other 24-hour period so that over a two-week period, each day of the week was sampled.

Industry: Mobil Oil Corporation

Treatment Facilities

The treatment facilities at Mobil consist of three separate discharge systems--the North Pond, the Channelized Pond, and the Commissioner's Ditch. Each of the systems has some type of oil separation and skimming equipment installed. All process water, once-through cooling water, cooling tower blowdown, boiler blowdown, surface runoff and ballast water from incoming ships discharge through one or more of the three systems. All the systems discharge directly to the Delaware River following oil separation.

Sampling Site and Equipment

During the sampling program, flows were not measured at any of the three outfall systems. For this reason, separate composite samples were taken at each of the three outfalls and then combined to make up a total composite based on flows estimated by Mobil personnel.

North Pond: The sampling point at the North Pond was at the outfall structure. A Protec Model sampler was employed at this point.

Channelized Pond: The sampling point was at the outfall structure using a Protec Model sampler.

Commissioner's Ditch: The elevation of the outfall structure at Commissioner's Ditch is such that discharges occur only at low tides. For this reason, a composite sampler was utilized at this point in which one-hour composites were obtained continuously and then composited based on tidal time charts.

Sampling Program

Twenty-four hour composites were taken once a week at each of the outfall systems. This type of sampling program was necessitated because of the absence of reliable flow data and the exigency of scheduling samplers. Additional data was supplied by another consultant conducting an in-plant survey for Mobil.

Industry: Houdry Process and Chemical Co.

Treatment Facilities

The treatment facilities at Houdry consist of two separate systems, the once-through cooling water system and the organic wastewater system. Two separate lift stations pump the waste streams to a common manhole on Mobil property. The combined waste then outfalls to a surface ditch leading off of Mobil property to the Delaware River. Included in the organic wastewater system is the septic tank overflow and cooling tower blowdown.

Sampling Sites and Equipment

Since there are two waste systems, two sampling sites were selected, each at their respective lift station. In both cases, sampling cocks on the discharge side of the pumps were connected to collection containers. When the pumps were operating, a steady stream of waste entered the containers. Composite samples, therefore, were obtained according to flows. Samples collected at the once-through cooling water lift station were analyzed separately to determine if any outside contamination was present.

Sampling Program

Twenty-four hour composites of both streams were taken initially. After several analyses of the cooling water waste, only grab samples were taken. Composites on the organic waste stream were continued at a frequency of three per week.

Industry: Hercules, Inc.

Treatment Facilities

The treatment facilities at Hercules consist of neutralization, equalization, extended aeration, clarification and chlorination. The outfall from the chlorine contact tank is routed directly to the Delaware River. Included within the wastewater stream is some process area surface runoff. A separate, highly concentrated waste consisting of spent sodium carbonate is incinerated in a thermal oxidizer unit.

Sampling Equipment and Site

A sampling site was established at the neutralization facility. This facility is well mixed and acts as a wet well for the lift station which pumps the waste to the biological treatment facilities. Samples were taken from a sampling cock on the discharge side of the lift pump and composited in a 55 gallon drum over a 24-hour period.

A second sampling point was established at the thermal oxidizer unit as this waste might be discharged into the regional system. A sampling cock on the discharge side of the recirculation pump at the feed storage tank was employed to obtain grab samples.

Sampling Program

Twenty-four hour composites of the organic waste stream were collected every other 24-hour period. Grab samples of the waste discharged to the thermal oxidizer were taken at various intervals to establish the organic strength of the waste.

Industry: duPont-Repauno Works

Treatment Facilities

The present facilities at Repauno consist of an open ditch system that collects all the cooling water and organic waste streams. The waste is discharged directly into the Delaware River after neutralization and floatable solids separation.

Sampling Sites and Equipment

The waste segregation program within the Repauno Plant was not yet completed and therefore composites were taken manually from the three concentrated organic streams. Composites were based on the future waste segregation estimates.

Sampling Program

Composite samples were obtained three times per week.

Industry: Monsanto Co.

Treatment Facilities

The facilities at Monsanto consist of a lift station-force main system which outfalls directly to the Delaware River. All surface runoff is conveyed to the Delaware River via a separate system.

Sampling Site and Equipment

The sampling site selected at Monsanto was at the lift station and was the only place where a composite could be conveniently taken. A composite sampler was utilized such that a composite was collected every eight hours with a total composite made manually over a three-day collection period.

Sampling Program

Samples were composited three times a week; two of these composite samples were three-day composites and one sample was a one-day composite.

Industry: duPont - Carney's Point

Treatment Facilities

The facilities at Carney's Point consist of a lift station-force main system which discharges waste directly to the Delaware River. Most of the plant's aqueous waste is carried through this system.

Sampling Site and Equipment

The sampling site at Carney's Point was at the lift station and consists of an air-operated valve assembly on the discharge side of the lift pumps. The samples were composited in a stainless steel 55 gallon drum.

Sampling Program

Composite samples were obtained three times a week and depending on the pick-up date were either two or three-day composites.

Industry: duPont - Chambers Works

Treatment Facilities

The waste treatment facilities at the Chambers Works consist of a ditch system that outfalls into a sedimentation basin. The waste is then neutralized and pumped to the Delaware River.

Sampling Sites and Equipment

The waste segregation program had not been fully completed at the Chambers Works at the time of sampling. Therefore, composites of various organic streams were based on estimated waste discharges after the segregation program was completed.

Sampling Program

Composites of the projected waste streams were made daily.

The mode of sample pick-up for the Pilot Plant phase of the Project was varied from the sampling program previously described because of the daily wastewater volume requirement and the logistical problems involved. Although the sampling locations remained the same, industrial and municipal wastewater samples were collected from the participants and conveyed to the pilot plant site in 5,600 gallon capacity tank trucks. This, in effect, represents a "grab" rather than a "composite" approach in obtaining the samples. It should be recognized, however, that these samples were collected a minimum of twice weekly from each participant over a period of twelve months, which would imply a statistical significance.

WASTEWATER CHARACTERIZATION AND FLOW

The characterization of wastewaters received from the participants during all phases of this program and the respective volumes of flow are summarized herein.

Quantity

The samples collected from each industrial participant were composited according to stated design flows and subsequently pumped to the bench scale units. No municipal wastes were included in this phase of the investigation because of their minor contribution to the total input, both in terms of hydraulic and organic loading. The contributing percentages of flow were slightly altered when the pilot plant studies commenced because of an updating of effluent discharge volumes obtained from the participants. The basis for compositing participant wastewater contributions for the bench scale studies and the revised formula for equalizing the contributions for the pilot plant study are summarized in Table I. The flow distribution is stated in terms of estimated 1975 values.

Quality

Detailed analyses of the industrial wastewaters were begun as soon as the sampling systems were installed at the individual plant sites. The analytical program for the integrated industrial waste sample was begun after all sampling systems were completed and the estimated flows for 1975 had been received. Initially, 24-hour composite samples from each plant were analyzed approximately three times a week.

The integrated sample was analyzed once a week, and this schedule was continued until the pilot plant was operating on the integrated waste stream. At that time, the Task P-1, or bench scale, analytical program was replaced by the pilot plant evaluation program.

Analyses were also performed on five separate municipal wastewaters.

Three 24-hour composite samples were collected at each of the following plants and analyzed for the same constituents as the industrial wastewaters: Pennsville, Salem City, Upper Penns Neck, Woodbury, and Paulsboro. These characterization data are reported in Chapter V of the Final Preliminary Engineering and Feasibility Study submitted by Engineering-Science in June, 1970.

Procedures

The analyses performed on the individual samples and the methods used are as follows:

1. pH was measured with a Leeds and Northrup pH meter.
2. Alkalinity, acidity and neutralization determinations were made with a Fisher Automatic Tritrimeter with 0.02 N sodium hydroxide or 0.02 N sulfuric acid.
3. Chemical oxygen demand (COD) was measured in accordance with Standard Methods using the 10.0 ml alternate procedure.
4. Biochemical oxygen demand (BOD) was determined in accordance with Standard Methods using seed acclimatized to the individual industrial wastewaters.
5. Dissolved oxygen was measured with a Weston-Stack D.O. meter. The meter was calibrated daily using the Winkler Method.
6. Nitrate and nitrite determinations were made with a Technicon Autoanalyzer in accordance with the Technicon Manual.
7. Total Kjeldahl nitrogen was measured with a Technicon Auto-analyzer in accordance with the Technicon Manual.

8. Total phosphorus was measured in accordance with Standard Methods.

9. Phenol was measured in accordance with Standard Methods except that a 100 ml sample was distilled instead of 500 ml.

10. All solids measurements were in accordance with Standard Methods.

11. Methylene Blue Active Substances (MBAS) were measured in accordance with the Water and Sewage Analysis Methods Manual, Hach Chemical Company, using the methyl green procedure.

12. All heavy metals were measured using a Perkin-Elmer Atomic Absorption Spectrophotometer Model 303.

13. Total organic carbon (TOC) was measured with a Beckman Model 915 Total Organic Carbon Analyzer.

14. Color determinations were made with a color comparator in accordance with Standard Methods, 12 Edition (1965).

Data Handling and Output

Raw data were transferred from laboratory work sheets to standardized data sheets, with each sample identified only by a three digit code and the date. From the standardized sheets, the data were transferred to computer cards, and then read and stored on discs by an IBM 360 Computer.

The output from the computer consisted of the following for each individual wastewater: one sheet presenting all data to date and summarizing each constituent in terms of high value, low value, average, and the standard deviation based on N observations; a second sheet with the ratios BOD_5: COD, BOD_5: TOD, TOC: COD; a third sheet summarizing flow data.

Results

The computer output sheets are not included in this report and are tabulated separately as task reports because of the bulk of information accumulated during this project. However, a statistical water quality representation of the samples received from each participant during the bench scale phase of the project is tabulated in Table 2. A similar presentation of the quality data observed during the pilot plant studies is given in Table 3. The characteristics of the combined industrial and municipal samples used in the bench scale phase are summarized in Table 4. The combined characterization data of the wastewaters applied to the pilot plant, effectively representing the quality of water which would have to be

TABLE 1

PARTICIPANT WASTEWATER FLOWS

PARTICIPANT	Five-Year Projected Flow (MGD)*	Percent Contribution	Revised Five-Year Flow (MGD)**	Revised Percent Contribution
duPont - Chambers Works	38.60	43.90	45.21	54.50
Mobil	26.00	29.60	14.00	16.87
Texaco	8.60	9.70	6.80	8.20
Shell	3.00	3.40	3.00	3.61
Monsanto	3.00	3.40	3.25	3.92
duPont - Carney's Point	2.40	2.70	3.18	3.83
Goodrich	1.20	1.40	1.30	1.57
duPont - Repauno	1.10	1.30	0.25	0.30
Houdry	0.30	0.40	0.25	0.30
Hercules	0.14	0.20	0.16	0.20
Municipalities	3.50	4.00	5.57	6.70
	87.84 MGD	100.00%	82.97 MGD	100.00%

* Basis for conducting bench scale studies.
** Basis for conducting pilot plant studies.

TABLE 2(A)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 015*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	10.6	8.3	9.4	0.7	0.07	16
TDS,mg/1	42,570	15,760	23,497	6,792	0.30	18
VDS, mg/1	9,440	800	3,154	2,513	0.80	17
TSS, mg/1	560	10	80	125	1.54	17
VSS, mg/1	70	0.01	31	17	0.55	16
COD, mg/1	28,000	7,600	14,977	5,542	0.37	18
BOD_5, mg/1	13,200	2,108	7,463	2,655	0.36	18
TOC, mg/1	7,175	2,411	4,656	1,465	0.31	16
TKN, mg/1	30.0	2.5	14.0	7.5	0.54	17
Total P, mg/1	5.0	0.3	1.8	1.1	0.61	17
Phenols, mg/1	5,250.0	800.0	2,187.5	1,188.7	0.54	12
MBAS, mg/1	12.2	4.8	8.5	3.7	0.44	2
Cr, mg/1**	0.6	0.2	0.4	0.1	0.25	18
Cu, mg/1**	0.3	< 0.1	< 0.2	-	-	18
Fe, mg/1**	6.8	< 0.1	< 0.35	-	-	18
Ni, mg/1**	0.9	< 0.1	< 0.4	-	-	18
Pb, mg/1**	1.8	0.2	0.5	0.4	0.80	18
Zn, mg/1**	0.6	< 0.1	< 0.2	-	-	18

* Composited samples

** Sensitivity of analysis = 0.1 mg/1

TABLE 2(B)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 025*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	9.6	2.7	7.0	4.4	0.63	21
TDS, mg/l	2,860	210	920	599	0.65	26
VDS, mg/l	1,500	80	386	286	0.74	26
TSS, mg/l	140	1	31	32	1.03	24
VSS, mg/l	120	1	23	26	1.13	24
COD, mg/l	930	107	238	146	0.61	32
BOD_5, mg/l	98	16	57	20	0.35	24
TOC, mg/l	156	21	48	30	0.62	26
TKN, mg/l	34.0	1.0	16.5	8.3	0.50	24
Total P, mg/l	32.0	0.1	3.6	7.1	1.97	25
Phenols, mg/l	6.8	0.0	1.6	1.8	1.13	21
MBAS, mg/l	3.8	0.4	1.2	1.3	1.08	5
Cr, mg/l**	1.1	< 0.1	< 0.3	-	-	24
Cu, mg/l**	< 0.1	< 0.1	< 0.1	-	-	24
Fe, mg/l**	3.2	< 0.1	< 1.0	-	-	24
Ni, mg/l**	0.7	< 0.1	< 0.2	-	-	24
Pb, mg/l**	1.6	< 0.1	< 0.3	-	-	24
Zn, mg/l**	1.1	< 0.1	< 0.2	-	-	24

* Composited samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 2 (C)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 035*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	8.6	3.3	6.5	1.7	0.26	10
TDS, mg/1	1,110	340	700	212	0.30	12
VDS, mg/1	520	1	209	131	0.63	12
TSS, mg/1	60	1	38	19	0.50	11
VSS, mg/1	60	1	27	19	0.70	11
COD, mg/1	461	203	290	101	0.35	12
BOD_5, mg/1	110	47	66	17	0.26	10
TOC, mg/1	116	11	52	39	0.75	10
TKN, mg/1	40.0	3.2	21.2	12.6	0.59	10
Total P, mg/1	2.6	0.3	1.0	0.6	0.60	11
Phenols, mg/1	7.9	0.0	2.8	2.8	1.0	9
MBAS, mg/1	40.5	5.1	15.5	12.8	0.83	5
Cr, mg/1**	0.3	< 0.1	< 0.2	-	-	11
Cu, mg/1**	< 0.1	< 0.1	< 0.1	-	-	11
Fe, mg/1**	1.9	0.2	0.6	-	-	11
Ni, mg/1**	0.2	< 0.1	< 0.1	-	-	11
Pb, mg/1**	3.5	< 0.1	< 0.5	-	-	11
Zn, mg/1**	0.4	< 0.1	< 0.2	-	-	11

* Composited samples

** Sensitivity of analysis = 0.1 mg/1

TABLE 2(D)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 045*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	4.2	2.0	2.7	0.5	0.18	20
TDS, mg/l	3,750	1,500	2,446	752	0.31	19
VDS, mg/l	1,320	30	769	354	0.46	19
TSS, mg/l	120	1	54	38	0.70	18
VSS, mg/l	90	1	32	27	0.84	18
COD, mg/l	927	251	495	176	0.36	20
BOD_5, mg/l	300	41	181	60	0.33	18
TOC, mg/l	233	79	133	39	0.29	20
TKN, mg/l	39.0	1.0	17.8	11.4	0.64	14
Total P, mg/l	55.0	0.3	8.6	11.6	1.35	19
Phenols, mg/l	6.9	1.0	4.1	1.5	0.37	10
MBAS, mg/l	9.5	4.6	6.3	2.3	0.37	3
Cr, mg/l**	3.5	< 0.1	< 1.1	-	-	12
Cu, mg/l**	1.1	0.3	0.6	0.2	0.33	12
Fe, mg/l**	5.0	2.3	3.6	0.8	0.22	12
Ni, mg/l**	0.5	< 0.1	< 0.2	-	-	12
Pb, mg/l**	1.0	< 0.1	< 0.5	-	-	12
Zn, mg/l**	5.8	0.8	2.3	1.5	0.65	12

* Composited samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 2(E)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 065*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	2.6	1.2	1.9	0.4	0.21	22
TDS, mg/l	4,600	1,980	3,423	1,086	0.32	3
VDS, mg/l	1,720	1,080	1,327	281	0.21	3
TSS, mg/l	200	1	60	81	1.35	4
VSS, mg/l	180	1	53	74	1.40	4
COD, mg/l	1,788	392	767	319	0.42	23
BOD_5, mg/l	780	120	329	140	0.43	22
TOC, mg/l	480	129	238	98	0.41	20
TKN, mg/l	38.0	0.8	8.3	9.6	1.16	23
Total P, mg/l	2.0	0.0	0.6	0.4	0.67	23
Phenols, mg/l	25.0	0.0	3.7	8.7	2.34	7
MBAS, mg/l	4.8	0.2	1.7	2.2	1.29	3
Cr, mg/l**	0.6	< 0.1	< 0.2	-	-	18
Cu, mg/l**	0.3	< 0.1	< 0.1	-	-	18
Fe, mg/l**	45.0	3.8	8.1	9.1	1.12	18
Ni, mg/l**	0.8	< 0.1	< 0.2	-	-	18
Pb, mg/l**	0.9	< 0.1	< 0.2	-	-	18
Zn, mg/l**	1.2	0.2	0.4	0.2	0.50	18

* Composited samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 2(F)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 075*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	11.4	2.7	8.7	2.3	0.26	18
TDS, mg/l	5,680	190	1,446	1,487	1.03	18
VDS, mg/l	1,970	110	623	540	0.87	18
TSS, mg/l	3,130	20	312	717	2.30	17
VSS, mg/l	360	1	56	83	1.48	16
COD, mg/l	154	39	88	35	0.40	18
BOD_5, mg/l	69	3	14	17	1.21	15
TOC, mg/l	68	4	30	17	0.57	16
TKN, mg/l	1,250.0	0.7	194.5	397.3	2.04	14
Total P, mg/l	92.8	0.3	7.1	22.2	3.13	16
Phenols, mg/l	2.9	0.0	0.5	0.9	1.80	10
MBAS, mg/l	1.3	0.2	0.7	0.5	0.71	3
Cr, mg/l**	0.4	< 0.1	< 0.1	-	-	13
Cu, mg/l**	< 0.1	< 0.1	< 0.1	-	-	13
Fe, mg/l**	1.2	< 0.1	< 0.2	-	-	13
Ni, mg/l**	0.2	< 0.1	< 0.1	-	-	13
Pb, mg/l**	0.9	< 0.1	< 0.2	-	-	13
Zn, mg/l**	0.3	< 0.1	< 0.1	-	-	13

* Composited samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 2(G)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 085*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	10.3	3.6	7.9	1.9	0.24	9
TDS, mg/1	1,590	580	902	267	0.30	13
VDS, mg/1	1,200	140	471	317	0.67	13
TSS, mg/1	60	1	20	18	0.90	13
VSS, mg/1	50	1	11	15	1.36	12
COD, mg/1	1,092	4	400	290	0.73	27
BOD_5, mg/1	510	1	160	114	0.71	22
TOC, mg/1	249	7	96	64	0.67	23
TKN, mg/1	14.5	0.1	2.6	3.9	1.5	11
Total P, mg/1	1.2	0.4	0.8	0.3	0.38	12
Phenols, mg/1	3.4	0.1	0.9	1.0	1.11	8
MBAS, mg/1	1.0	1.0	1.0	-	-	1
Cr, mg/1**	1.6	< 0.1	< 0.9	-	-	13
Cu, mg/1**	< 0.1	< 0.1	< 0.1	-	-	13
Fe, mg/1**	1.1	< 0.1	< 0.3	-	-	13
Ni, mg/1**	0.4	< 0.1	< 0.2	-	-	13
Pb, mg/1**	0.5	< 0.1	< 0.3	-	-	13
Zn, mg/1**	1.5	< 0.1	< 0.3	-	-	13

* Composited samples

** Sensitivity of analysis = 0.1 mg/1

TABLE 2(H)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 095*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	10.2	1.3	3.5	3.0	0.86	16
TDS, mg/1	26,800	8,560	17,831	5,263	0.30	13
VDS, mg/1	19,980	4,730	11,590	4,428	0.38	13
TSS, mg/1	17,270	116	5,640	4,825	0.86	13
VSS, mg/1	17,270	40	5,548	4,839	0.87	13
COD, mg/1	43,300	2,500	19,186	10,912	0.57	22
BOD_5,mg/1	25,000	2,409	10,062	6,298	0.62	22
TOC, mg/1	15,595	2,391	5,896	4,638	0.79	14
TKN, mg/1	4,400	9	897	1,248	1.39	15
Total P, mg/1	305.0	4.0	115.8	88.3	0.76	15
Phenols, mg/1	320.0	0.5	80.7	91.1	1.13	9
MBAS, mg/1	710.0	104.0	405.3	247.4	0.61	3
Cr, mg/1**	0.7	0.4	0.5	0.1	0.20	13
Cu, mg/1**	30.9	2.4	20.4	6.5	0.32	13
Fe, mg/1**	10.8	< 0.1	< 3.4	-	-	13
Ni, mg/1**	10.7	0.3	1.3	2.7	2.08	13
Pb, mg/1**	3.5	< 0.1	< 0.6	-	-	13
Zn, mg/1**	0.5	< 0.1	< 0.2	-	-	13

* Composited samples

** Sensitivity of analysis = 0.1 mg/1

TABLE 2(I)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
INDUSTRIAL WASTEWATER 105 *

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	12.2	4.7	10.8	2.1	0.19	11
TDS, mg/l	19,700	9,220	13,201	2,445	0.19	14
VDS, mg/l	9,300	1,410	4,266	2,275	0.53	14
TSS, mg/l	2,620	200	1,026	545	0.53	14
VSS, mg/l	1,090	100	410	240	0.59	14
COD, mg/l	5,200	1,590	3,363	1,009	0.30	26
BOD_5, mg/l	2,922	115	1,795	600	0.33	24
TOC, mg/l	1,994	828	1,335	323	0.24	22
TKN, mg/l	22.0	1.1	7.3	5.5	0.75	11
Total P, mg/l	4.0	0.1	1.0	1.2	1.20	12
Phenols, mg/l	12.0	0.2	2.7	4.2	1.56	6
MBAS, mg/l	1.3	1.0	1.2	0.1	0.08	3
Cr, mg/l**	3.4	1.1	2.2	0.6	0.27	12
Cu, mg/l**	< 0.1	< 0.1	< 0.1	-	-	12
Fe, mg/l**	4.5	< 0.1	< 1.1	-	-	12
Ni, mg/l**	0.6	0.2	0.4	0.1	0.25	12
Pb, mg/l**	1.0	0.3	0.6	0.2	0.33	12
Zn, mg/l**	0.8	< 0.1	< 0.2	-	-	12

* Composited samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 2(J)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
BENCH SCALE PHASE
MUNICIPAL WASTEWATERS*

PARAMETER**	PENNSVILLE	UPPER PENNS NECK	SALEM	PAULSBORO	WOODBURY
TDS	480	490	480	460	450
TSS	40	30	25	70	70
COD, mg/l	300	420	300	547	365
BOD_5, mg/l	119	123	97	185	110
$NO_2 + NO_3$ Nitrogen, mg/l	0.6	0.7	0.5	1.0	1.7
TKN, mg/l	16.6	23.7	12.9	20.4	12.9
Total P, mg/l	13.5	15	10	12	10
Cr, mg/l	< 1.0	< 1.0	< 1.0	< 1.0	< 1.0
Cu, mg/l	< 0.25	< 0.25	< 0.25	< 0.25	< 0.25
Fe, mg/l	< 1.0	< 1.0	< 1.0	< 1.0	< 1.0
Ni, mg/l	< 2.5	< 2.5	< 2.5	< 2.5	< 2.5
Pb, mg/l	< 2.0	< 2.0	< 2.0	< 2.0	< 2.0
Zn, mg/l	< 1.0	< 1.0	< 1.0	< 1.0	< 1.0

* Composited samples

** Represent mean values

TABLE 3(A)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 015*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	11.9	4.3	8.59	1.58	0.18	58
COD, mg/1	90,200	189	16,791	17,389	1.04	44
TOC, mg/1	6,300	763	3,560	1,578	0.44	15
TOD, mg/1	77,000	380	13,963	13,734	0.98	43
Cu, mg/1**	0.3	<0.1	<0.14	-	-	9
Cr, mg/1**	2.8	0.2	1.1	.8	.73	9
Ni, mg/1**	0.5	<0.1	<0.22	-	-	9
Zn, mg/1**	0.9	0.2	0.43	0.23	0.53	9
Pb, mg/1**	0.6	<0.1	<0.17	-	-	9
Fe, mg/1**	20.0	4.4	7.86	4.87	0.62	9
Mn, mg/1**	0.4	<0.1	<0.24	-	-	5
Ag, mg/1**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/1**	<0.1	<0.1	<0.1	-	-	5
Hg, mg/1	0.0190	0.0001	0.00478	0.00668	1.40	7

* Grab type samples

** Sensitivity limit of analysis = 0.1 mg/1

TABLE 3(B)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 025*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	11.7	1.9	8.72	1.47	0.17	193
COD, mg/l	12,800	81	542	1,054	1.94	150
TOC, mg/l	429	25	128.7	96.1	0.75	34
TOD, mg/l	3,500	42	456.0	408.8	0.90	163
Cu, mg/l**	0.3	<0.1	<0.11	-	-	47
Cr, mg/l**	1.3	<0.1	<0.40	-	-	47
Ni, mg/l**	<0.1	<0.1	<0.10	-	-	47
Zn, mg/l**	1.3	<0.1	<0.30	-	-	47
Pb, mg/l**	0.2	<0.1	<0.11	-	-	47
Fe, mg/l**	8.2	<0.1	<1.73	-	-	47
Mn, mg/l**	0.9	<0.1	<0.14	-	-	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	0.6	0.3	0.46	0.15	.33	5
Hg, mg/l	0.0048	0.0001	0.00129	0.00122	0.94	44

* Grab type samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 3(C)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 033*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS
pH	12.1	2.3	7.73	2.07	0.27	253
COD, mg/l	9,280	56	754	1,212	1.61	204
TOC, mg/l	822	17	200.2	189.1	0.94	54
TOD ,mg/l	12,500	30	630	1,140	1.81	213
Cu, mg/l**	0.3	<0.1	<0.11	-	-	47
Cr, mg/l**	16.6	<0.1	<0.60	-	-	47
Ni, mg/l**	0.2	<0.1	<0.10	-	-	47
Zn, mg/l**	20.0	<0.1	<0.80	-	-	47
Pb, mg/l**	0.3	<0.1	<0.13	-	-	47
Fe, mg/l**	20.0	0.2	2.18	3.24	1.49	47
Mn, mg/l**	0.6	0.2	0.36	0.02	0.06	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	0.2	0.2	0.20	0.00	0.00	5
Hg, mg/l	0.0060	0.0001	0.00139	0.00171	1.23	32

* Grab type samples

** Sensitivity of analysis - 0.1 mg/l

TABLE 3(D)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 034*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	11.0	1.9	6.51	1.96	0.30	225
COD, mg/l	38,100	48	687	3,069	4.47	178
TOC, mg/l	218	12	50.3	43.3	0.86	45
TOD, mg/l	4,625	30	273.3	374.6	1.37	192
Cu, mg/l**	0.9	<0.1	<0.14	-	-	53
Cr, mg/l**	1.4	<0.1	<0.37	-	-	53
Ni, mg/l**	0.2	<0.1	<0.10	-	-	53
Zn, mg/l**	2.0	<0.1	<0.38	-	-	53
Pb, mg/l**	0.2	<0.1	<0.11	-	-	53
Fe, mg/l**	25.0	<0.1	<2.15	-	-	53
Mn, mg/l**	1.0	0.2	0.5	0.33	0.66	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	0.2	<0.1	<0.16	-	-	5
Hg, mg/l	0.0143	0.0001	0.00224	0.00319	1.42	40

* Grab type samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 3(E)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 045*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	5.1	0.7	2.48	0.37	0.15	132
COD, mg/l	884	16	515.1	120.0	0.23	111
TOC, mg/l	416	416	416	-	-	1
TOD, mg/l	850	63	407.6	170.1	0.42	107
Cu, mg/l**	1.0	<0.1	<0.47	-	-	35
Cr, mg/l**	2.4	<0.1	<0.56	-	-	35
Ni, mg/l**	0.5	<0.1	<0.11	-	-	35
Zn, mg/l**	8.6	0.2	2.32	2.18	0.94	35
Pb, mg/l**	0.4	<0.1	<0.21	-	-	35
Fe, mg/l**	20.0	3.2	6.70	3.39	0.50	35
Mn, mg/l**	2.5	0.7	1.16	0.81	0.70	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	0.3	<0.1	<0.22	-	-	5
Hg, mg/l	0.1635	0.0002	0.00851	0.02374	2.79	47

* Composite samples based on flow

** Sensitivity of analysis = 0.1 mg/l

TABLE 3(F)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 055*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	10.4	6.5	8.80	1.14	0.13	24
COD. mg/1	5,700	760	1,896	1,276	0.67	13
TOC, mg/1	480	320	400	110	0.28	2
TOD, mg/1	2,050	300	997.4	368.1	0.37	19
Cu, mg/1**	0.2	<0.1	<0.11	-	-	15
Cr, mg/1**	0.4	<0.1	<0.21	-	-	15
Ni, mg/1**	<0.1	<0.1	<0.10	-	-	15
Zn, mg/1**	0.3	<0.1	<0.15	-	-	15
Pb, mg/1**	0.8	<0.1	<0.18	-	-	15
Fe, mg/1**	50.0	1.4	12.9	13.1	1.02	15
Mn, mg/1**	-	-	-	-	-	-
Ag, mg/1**	-	-	-	-	-	-
Sr, mg/1**	-	-	-	-	-	-
Hg, mg/1	0.0058	0.0001	0.00193	0.00187	0.97	12

* Grab type samples

** Sensitivity of analysis = 0.1 mg/1

TABLE 3(G)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 065*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	9.2	0.9	2.07	1.24	0.60	102
COD, mg/l	5,780	310	1,026	887	0.86	71
TOC, mg/l	3,364	68	688.8	874.1	1.27	16
TOD, mg/l	1,310	22	222.9	239.5	1.07	41
Cu, mg/l**	0.4	<0.1	<0.20	-	-	20
Cr, mg/l**	0.5	<0.1	<0.22	-	-	20
Ni, mg/l**	<0.1	<0.1	<0.10	-	-	20
Zn, mg/l**	1.0	<0.1	<0.43	-	-	20
Pb, mg/l**	0.9	<0.1	<0.16	-	-	20
Fe, mg/l**	725.0	0.4	47.6	159.6	3.35	20
Mn, mg/l**	0.5	0.2	0.32	0.13	0.41	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	0.4	0.2	0.32	0.08	0.25	5
Hg, mg/l	0.0047	0.0008	0.00247	0.00128	0.52	17

* Grab type samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 3(H)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 085*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	10.2	2.2	7.40	1.63	0.22	95
COD, mg/l	1,582	23	321.2	321.4	1.00	75
TOC, mg/l	2,080	1	172.5	456.1	2.64	20
TOD, mg/l	4,360	10	202.6	499.9	2.47	78
Cu, mg/l**	0.4	<0.1	<0.14	-	-	22
Cr, mg/l**	1.1	<0.1	<0.57	-	-	22
Ni, mg/l**	0.2	<0.1	<0.11	-	-	22
Zn, mg/l**	0.7	<0.1	<0.28	-	-	22
Pb, mg/l**	0.8	<0.1	<0.14	-	-	22
Fe, mg/l**	11.3	<0.1	<2.54	-	-	22
Mn, mg/l**	1.1	<0.1	<0.32	-	-	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	0.4	<0.1	<0.28	-	-	5
Hg, mg/l	0.1080	0.0002	0.00748	0.02439	3.26	19

* Grab type samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 3(I)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 095*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	12.2	1.3	5.18	3.37	0.65	51
COD, mg/1	99,999	512	23,100	27,360	1.18	39
TOC, mg/1	17,985	118	7,875	6,689	0.85	10
TOD, mg/1	58,200	32	9,600	11,230	1.17	40
Cu, mg/1**	0.5	<0.1	<0.19	-	-	8
Cr, mg/1**	0.7	<0.1	<0.21	-	-	8
Ni, mg/1**	0.2	<0.1	<0.11	-	-	8
Zn, mg/1**	1.5	<0.1	<0.39	-	-	8
Pb, mg/1**	0.5	<0.1	<0.22	-	-	8
Fe, mg/1**	20.0	0.2	6.38	7.29	1.14	8
Mn, mg/1**	0.4	<0.1	<0.25	-	-	4
Ag, mg/1**	<0.1	<0.1	<0.1	-	-	4
Sr, mg/1**	0.8	0.2	0.37	0.28	0.76	4
Hg, mg/1	0.0810	0.0125	0.03542	0.02964	0.84	6

* Grab type samples

** Sensitivity of analysis = 0.1 mg/1

TABLE 3(J)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
INDUSTRIAL WASTEWATER 105*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	12.4	1.4	9.75	3.49	0.36	82
COD, mg/l	10,100	19	2,904	1,534	0.53	73
TOC, mg/l	2,312	56	815.4	576.4	0.71	14
TOD, mg/l	17,000	75	2,668	2,414	0.90	66
Cu, mg/l**	0.3	<0.1	<0.13	-	-	18
Cr, mg/l**	8.6	<0.1	<2.04	-	-	18
Ni, mg/l**	0.4	<0.1	<0.18	-	-	18
Zn, mg/l**	16.2	<0.1	<1.38	-	-	18
Pb, mg/l**	1.4	<0.1	<0.35	-	-	18
Fe, mg/l**	75.0	<0.1	<19.07	-	-	18
Mn, mg/l**	1.1	<0.1	<0.56	-	-	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	3.7	<0.1	<1.00	-	-	5
Hg, mg/l	0.0200	0.0005	0.00334	0.00483	1.45	16

* Grab type samples

** Sensitivity of analysis = 0.1 mg/l

TABLE 3(K)

CHARACTERIZATION OF PARTICIPANT WASTEWATERS
PILOT PLANT PHASE
MUNICIPAL WASTEWATER 835*

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
pH	9.8	2.5	7.39	0.70	0.09	203
COD, mg/l	2,690	71	393.7	292.5	0.74	178
TOC, mg/l	352	13	91.9	73.9	0.80	34
TOD, mg/l	1,900	20	280.6	221.2	0.79	175
Cu, mg/l**	0.3	<0.1	<0.12	-	-	43
Cr, mg/l**	0.5	<0.1	<0.16	-	-	43
Ni, mg/l**	<0.1	<0.1	<0.10	-	-	43
Zn, mg/l**	0.5	<0.1	<0.17	-	-	43
Pb, mg/l**	0.4	<0.1	<0.13	-	-	43
Fe, mg/l**	9.3	0.4	1.54	1.52	0.99	43
Mn, mg/l**	0.2	<0.1	<0.14	-	-	5
Ag, mg/l**	<0.1	<0.1	<0.1	-	-	5
Sr, mg/l**	0.3	<0.1	<0.22	-	-	5
Hg, mg/l	0.0092	0.0001	0.00226	0.00262	1.16	24

* Grab type samples obtained from Upper Penn's Neck Wastewater Treatment Plant

** Sensitivity of analysis = 0.1 mg/l

TABLE 4

CHARACTERIZATION OF COMBINED INDUSTRIAL & MUNICIPAL WASTEWATERS
BENCH SCALE PHASE

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.*
pH	3.1	1.9	2.6	0.4	0.15	6
TDS, mg/l	3,250	1,900	2,466	435	0.18	8
VDS, mg/l	1,230	390	861	262	0.30	8
TSS, mg/l	90	1	18	28	1.56	8
VSS, mg/l	80	1	14	26	1.86	8
COD, mg/l	908	570	688	102	0.15	8
BOD_5, mg/l	340	170	293	53	0.18	7
TOC, mg/l	230	155	196	24	0.12	7
TKN, mg/l	39.6	3.0	18.1	15.0	0.83	7
Total P, mg/l	4.6	2.3	3.4	0.8	0.24	7
Phenols, mg/l	9.2	3.1	6.6	1.8	0.27	6
MBAS, mg/l	14.5	14.5	14.5	-	-	1
Cr, mg/l **	2.3	0.3	0.8	0.6	0.75	7
Cu, mg/l **	0.9	0.5	0.7	0.1	0.14	7
Fe, mg/l **	7.2	3.2	4.9	1.6	0.33	7
Ni, mg/l **	0.7	< 0.1	< 0.2	-	-	7
Pb, mg/l **	1.0	0.2	0.5	0.2	0.40	7
Zn, mg/l **	2.8	0.5	1.2	0.7	0.58	7

* Composited samples

** Sensitivity of analysis = 0.1 mg/l

treated in the prototype system, are tabulated in Table 5. The parameters cited are for samples obtained at the neutralization tank influent or primary clarifier effluent as noted in the Table. The statistical distribution of these key parameters are illustrated in Figures 2, 3, and 4.

DISCUSSION

The characterization data presented in this chapter represent the summarized results observed from the inception of the project in August, 1969, to the termination of the pilot plant study in October, 1971. Based on the frequency and number of observations accumulated during this time span, it was possible to accurately define the magnitudes and patterns of the pertinent constituents contained in the various wastewaters.

It is noted that there is some variation in the reported analyses from the individual participants in the bench scale phase, Table 2, and the pilot plant phase, Table 3. Although the sampling points were essentially the same, this variation can be attributed to in-plant modes of operation and the influence of the sampling date on process and cooling operations.

The coefficient of variation is indicative of the relative variations for each of the water quality parameters cited. For example, the coefficient of variation of the organic parameters (COD, BOD, TOC, and TOD) was generally higher for the individual industrial waste samples recorded during the pilot plant phase of the project than during the bench scale phase. This is reflected in Tables 4 and 5 and can be attributed to the respective number of observations and the nature of sampling. Conversely, a higher variation in suspended solids data was noted during the bench scale phase, which is reasonable when considering the nature of the test and the methods of obtaining the samples.

It is interesting to note that of the organic parameters listed in Tables 4 and 5, the COD and TOC variations as measured by the coefficient of variation were less than those for the BOD. This is most probably reflected by the accuracy of the tests, the COD and TOC analytical procedures being less subject to interferences and human error than the BOD test. The variations in suspended solids were higher than those reported for the organic parameters, although variations in dissolved solids concentrations were about the same. The coefficients of variation for phenols, nitrogen, phosphorus, and heavy metals followed no specific pattern, although the magnitudes approximate those reported from similar studies.

The distribution of organics, solids, and pH for the pilot plant influent water are illustrated in Figures 1, 2, and 3. The organic data presented in Figure 2 most probably represents two populations. For example, the organic concentration of the wastewaters is considerably lower in the summer than in the

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TABLE 5

CHARACTERIZATION OF COMBINED INDUSTRIAL AND MUNICIPAL WASTEWATERS PILOT PLANT PHASE

PARAMETER	HIGH	LOW	MEAN	STD. DEV.	COEF. OF VAR.	# OF OBS.
ph	5.0	2.0	2.9	0.6	0.21	36*
TDS, mg/l	3,182	1,275	1,742	408	0.23	24*
VDS, mg/l	1,182	290	576	204	0.35	24*
TSS, mg/l	100	28	55	20	0.36	36*
VSS, mg/l	80	20	44	14	0.32	36*
COD, mg/l	822	420	618	117	0.19	36*
BOD_5, mg/l	453	136	247	76	0.31	36*
TOC, mg/l	358	109	193	51	0.26	32*
TOD, mg/l	990	237	511	164	0.32	36*
Phenols, mg/l	18.75	0.75	7.39	3.22	0.44	54
Color, Std. Units**	1,800	200	794	364	0.46	82
Total P, mg/l**	13.0	0.2	2.0	2.0	1.0	182
TKN, mg/l**	45.0	10.5	25.8	7.4	0.29	181
Ammonia Nitrogen, mg/l**	36.5	4.0	23.7	6.5	0.31	140
$NO_2 + NO_3$, mg/l**	122.5	0.7	24.8	25.4	1.02	164
Cu, mg/l***	0.8	<0.1	0.38	-	-	39
Cr, mg/l***	1.9	<0.1	<0.46	-	-	39
Ni, mg/l***	0.2	<0.1	<0.1	-	-	30
Zn, mg/l***	7.8	<0.1	<2.11	-	-	39
Pb, mg/l***	0.4	<0.1	<0.17	-	-	30
Fe, mg/l***	10.2	<0.1	<4.52	-	-	39
Mn, mg/l***	1.8	<0.1	<0.73	-	-	39
Ag, mg/l***	<0.1	<0.1	<0.1	-	-	31
Sr, mg/l***	0.6	<0.1	<0.32	-	-	31
Hg, mg/l	0.0158	0.0000	0.00329	0.00407	1.24	27
Al, mg/l***	1.5	0.4	0.91	0.42	0.46	7
Cd, mg/l	0.03	< 0.01	< 0.02	-	-	6
SO_4, mg/l	592	416	502	71	0.14	7
MBAS, mg/l	9.0	3.0	5.8	2.2	0.38	8
Fecal Coliforms, No./100 ml	0	0	0	0	-	8

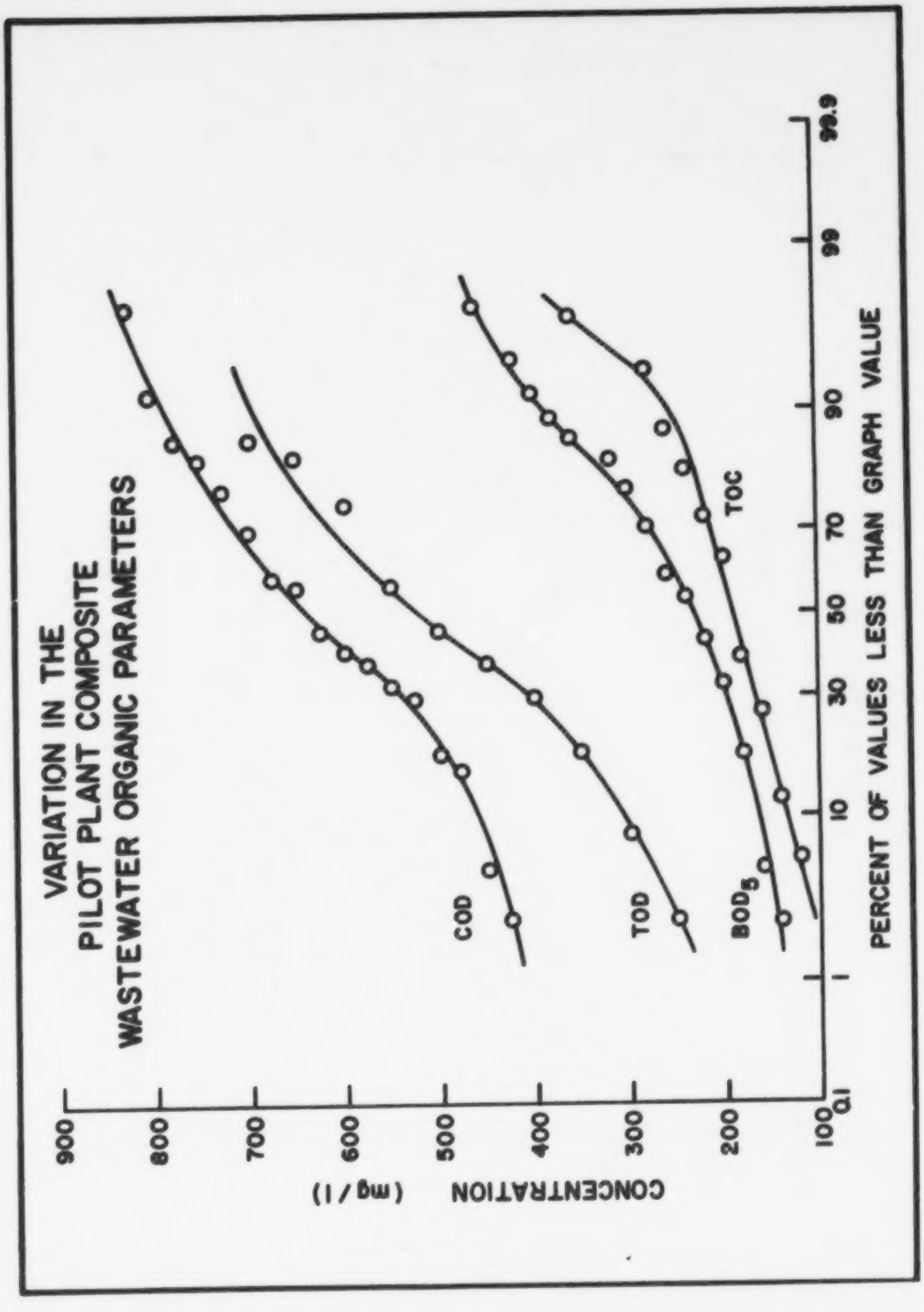
NOTE: All analyses were made on the raw wastewater except where noted.

* Number of weekly averages

** Values represent effluent from primary clarifier

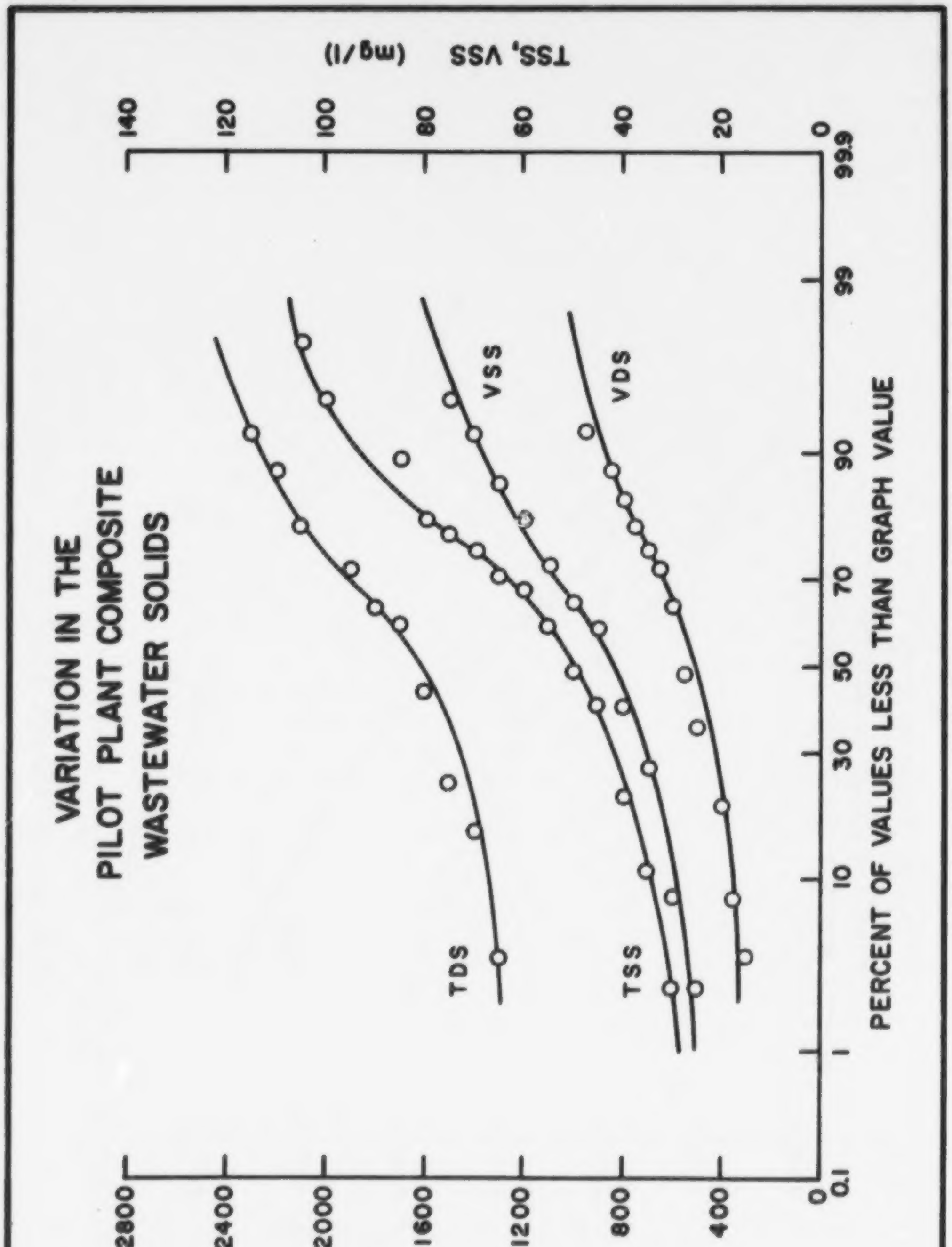
*** Sensitivity of analysis = 0.1 mg/l

Figure 2



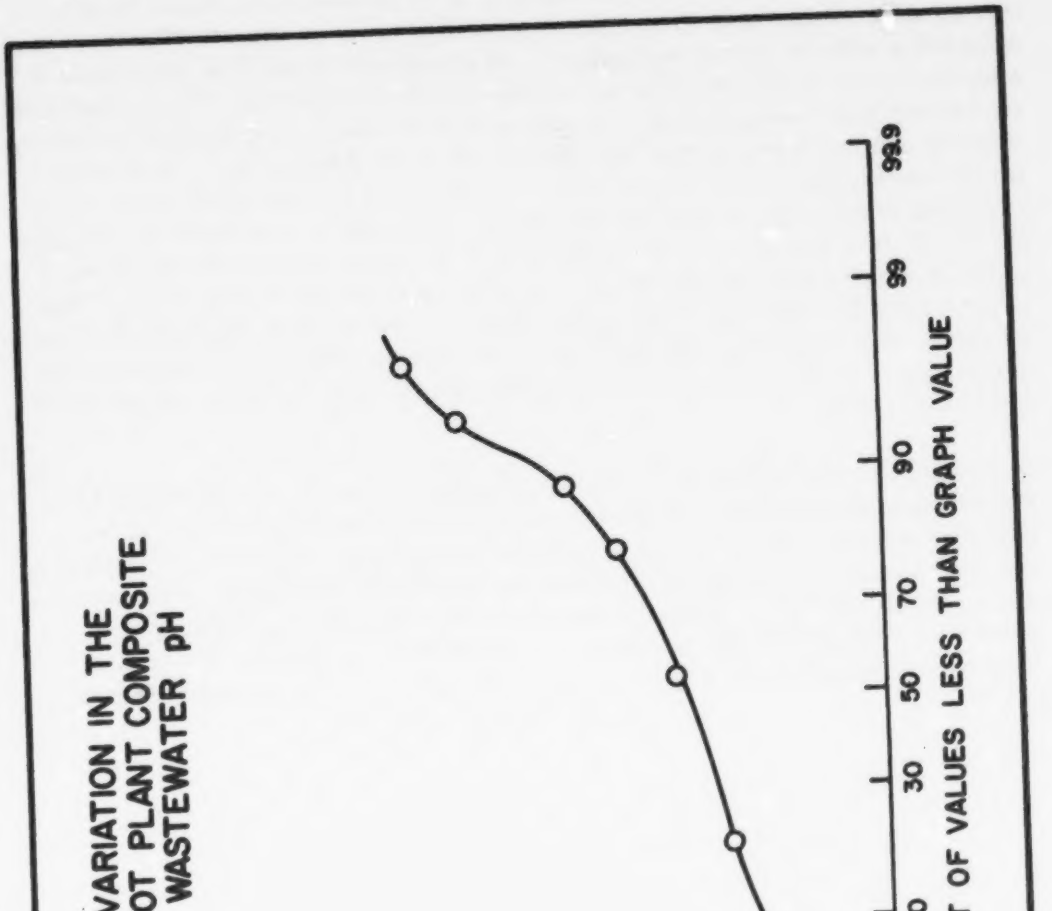
VARIATION IN THE PILOT PLANT COMPOSITE WASTEWATER ORGANIC PARAMETERS

Figure 3



VARIATION IN THE PILOT PLANT COMPOSITE WASTEWATER SOLIDS

Figure 4



VARIATION IN THE PILOT PLANT COMPOSITE WASTEWATER pH winter due to the volume of cooling water diluent present during the warm weather months. This is reflected in the probability curve geometry and should be considered when designing a waste treatment facility which is capable of producing an acceptable effluent during each season. In a practical sense, less importance is attached to the suspended solids distributions shown in Figure 3. This is predicated on the fact that the suspended solids observed in the pilot plant effluent following trucking and temporary storage are liable to be quite different from those in a wastewater discharge from the equalization facility at an individual plant and conveyed to the regional facility through an interceptor. As a matter of judgment, the levels shown in Figure 3 are considered to be conservative. The dissolved solids levels are more representative, however, and should be indicative of that expected for the combined wastewater influent to the regional treatment facility. As noted, there was much less variation in pH values, attributable in part to the dampening of batch dumps and surges by the equalization facilities preceding the pilot plant system.

In summary, certain patterns, both seasonal and operational, can be detected in the tabular and graphical presentation of the wastewater characterization data. Although one must recognize the constraints which are prevalent when interpreting this data (sampling methods and frequencies, analytical procedures, interferences, etc.), it still provides a rational approach for establishing an individual and collective characterization picture of the wastewaters involved. It is from this information that plant design, cost evaluation, and cost allocation were based.

SECTION V

BENCH SCALE TREATABILITY STUDIES

There are many aspects involved in the development of design criteria for wastewater treatment facilities through the use of bench scale and pilot scale treatability studies. The first logical step toward evaluating the treatability of a wastewater is the application of bench scale simulation techniques, observing system responses under various environmental and physical conditions.

There are several approaches which can be employed to evaluate the individual processes which comprise a total waste treatment system. It should be recognized, however, that regardless of the approach taken, the ultimate accuracy of the information developed from bench scale studies depends on several conditions, namely:

1. The characteristics of the wastewater used in the treatability tests are representative of those anticipated in the field;

2. The physical nature of the bench or pilot scale process is similar to the prototype unit;

3. Independent and dependent operational variables are considered; and,

4. Environmental parameters affecting process efficiency are defined. Observing these and other guidelines, bench and pilot scale simulation techniques can provide limited process information with respect to applicability, establishment of predictor relationships, and approximate determinations of process capacity. Although information garnered during these studies must be applied in a judicious manner, a treatability study which is properly programmed and carefully implemented does afford the basis for the logical development of unit process selection, design, and predictive performance.

OPERATIONAL PROCEDURES

The scope of the bench scale treatability program included an evaluation of pre- and primary treatment processes, secondary biological treatment, and ancillary studies related to sludge dewatering, chemical treatment, and physical treatment. The bench scale equipment consisted of standard laboratory glassware, commercially available testing equipment, and specially constructed process models. This equipment is described in the following sections of this Chapter.

The laboratory analytical schedule was programmed to provide sufficient data for adequately evaluating each of the processes considered. Analyses were performed using accepted analytical techniques, primarily conforming to Standard Methods, 12 Ed. (Reference 1). Many of the methods used in the treatability studies were based on those outlined in Water Pollution Control (Reference 2).

PRE- AND PRIMARY TREATMENT EVALUATION

Equalization

Experience has shown that treatment processes, whether physical, chemical, or biological, perform at a higher rate of efficiency if the hydraulic and organic load fluctuations to the process can be dampened. The most prevalent situations where the equalization principle should be applied are summarized as follows (Reference 3):

1. Biological Treatment

    A. Poisoning by high concentrations of toxic materials, even if only of slight duration.

    B. Inhibition by high concentrations of normally biodegradable materials.

    C. Short-term upsets caused by extreme deviations of input; transient effects.

2. Chemical Treatment - Variations in chemical demand, if not smoothed out, will require variable rate feeders, and sophisticated control systems.

3. Physical (equalization without treatment) - Where effluent regulations limit the concentration of a component in the discharge to a value which is above its long-term mean value, equalization facilities can smooth the concentration-time curve and attain compliance.

Equalization will occur in varying degrees at the plant site of each participant. Additionally, there will be some equalization in the conveyance system, and finally, equalization at the Regional plant can be instituted if considered necessary for adequate process performance. Although no bench scale equalization studies were conducted per se, a review of the individual modes of equalization and their influence with respect to regional treatment are discussed individually.

Equalization Basins at the Individual Plant Sites

Basin Size:

In essence, each industry will size its equalization basin based on the cost of buying more capacity in the interceptor sewer and treatment plant versus the cost of installing a larger equalization basin. Minimum interceptor costs would result by never exceeding the average annual flow rate by over 10 percent. However, if a particular industry experienced significant seasonal variations in wastewater flows, a very large equalization basin would be required and it might be more economical to buy more capacity in the regional system.

Minimum Basin Size:

The smallest equalization basin that an industry could economically consider would be designed only to store contaminated storm water runoff until it could be pumped into the system with the 10 percent allowable excess flow rate.

Using flow data presented in the Preliminary Engineering Report which was submitted in June, 1970, the size and detention time of the minimum equalization basin required for each industry pumping into the interceptor sewer is presented in Table 6. The minimum basin size is based on holding all of the runoff from a storm having five inches of precipitation in 24 hours.

With the exception of Hercules, which already has a relatively large equalization basin, the detention time provided by the minimum basin is rather small. A further consideration is that the operating volume in a small basin would normally be kept low so as to provide the maximum possible retention of storm water after a storm began. Therefore, the detention of process wastewaters provided by the minimum size equalization basins would normally be very small, and a negligible effect on equalizing fluctuations in the quality characteristics of the wastewaters would be expected.

Maximum Basin Size:

The maximum basin size would result from dampening out fluctuations in process wastewaters. Particularly critical would be seasonal fluctuations such as occur with industries having a large flow of contaminated, once-through cooling water. During the summer months, when the river temperature is high, more cooling water is required to achieve the same cooling effect that is obtained in the winter with considerably less water.

Depending on the amount of cooling water involved, it is indicated that an equalization basin having a detention time of between five and 10 days at the average yearly flow is required if seasonal flows are balanced using an excess pumping factor of 10 percent.

Currently, it is doubtful if five to 10 days equalization capacity will be economical and a practical maximum would probably be one to two days.

TABLE 6

EQUALIZATION BASIN SIZE BASED ON STORM WATER RUNOFF[a]

Location	1990 Flow MGD	Volume of Runoff[b] MG	Excess Allowable Pumping Rate MGD	Time to Pump All of Runoff Days	Detention of Process Wastewater Provided by Basin Equal to Runoff Volume Days
Monsanto	9.0	0.68	0.9	0.75	0.07
Repauno	7.2	1.36	0.72	1.90	0.19
Mobil	26.0	20.50	2.6	7.90	0.80
Houdry	0.4	0.14	0.04	3.50	0.35
Shell	6.0	0.68	0.6	1.10	0.11
Texaco	7.9	5.04	0.79	6.40	0.64
BFG	2.4	0.68	0.24	2.83	0.28

(a) Based on flows and runoff volumes from Table V-1, Task C-1 (Preliminary Interceptor Report)

(b) Based on five inches of rain in 24 hours

Effect of Equalization Basin on Wastewater Characteristics:

Assuming each industry will provide some equalization capacity to dampen out fluctuations in wastewater flows, there will be some effective equalization of the quality characteristics. The degree of equalization will depend on the capacity and the flow characteristics of the basin.

Two types of flow can occur in a basin: (1) plug flow, and (2) completely mixed flow. The relative amounts of these two types of flow plus the degree of short-circuiting and dead space that occurs in a basin determines the flow characteristics.

Ideally, a completely mixed basin without any dead space or short-circuiting would provide the highest degree of equalization of the fluctuations in the quality characteristics of a wastewater. In such a basin, the concentration of any constituent in the effluent from the basin would be the same as the concentrations within the basin. There would, therefore, be a maximum dampening of the fluctuations in influent quality characteristics.

Conversely, a basin with plug flow-regardless of the amount of dead space and short-circuiting-would provide little or no equalization of quality characteristics. Effluent concentrations would reflect those in the influent after the necessary time lag.

Although short-circuiting and dead space can be minimized by proper baffling and inlet and outlet structures, completely mixed systems are obtained only by providing external agitation. The cost of building and operating such a basin is therefore higher than for one with plug flow characteristics.

Because the equalization basin requirements at the individual plants are based solely on dampening out variations in flow rate, there is no economic incentive for installing a basin with completely mixed characteristics. The design of the basins will essentially be predicated on minimizing costs and will therefore have flow characteristics that are a combination of plug, mixed, short-circuiting, and dead space. The amount of mixing that does take place will be the result of wind action, thermal currents, inlet turbulence, etc., and essentially will be uncontrolled. Previous experiments have indicated that such basins usually have about 10 to 40 percent completely mixed characteristics.

Assuming the actual basins are approximately 25 percent completely mixed, with the remaining characteristics being divided equally among short-circuiting, plug flow, and dead space, the basins could effectively equalize fluctuations in concentrations that occur over a time interval equal to 25 percent of the theoretical detention time of the basin. Therefore, unless a participant constructs an unusually large equalization basin, only short term fluctuations in quality parameters will be equalized. Assuming the typical equalization capacity provided to dampen out wastewater flows is equal to two days, 25 percent of this, or approximately 12 hours, would be available to dampen out fluctuations in concentrations.

Equalization in Interceptor

The flow characteristics of the interceptor will be almost 100 percent plug flow. Some mixing will occur at the pumping stations, but this would be essentially negligible.

Because the flow in the interceptor will have plug characteristics, there will be no dampening out of the fluctuations in concentrations of the various constituents in the combined wastewaters. This can only occur in the individual equalization basins.

As each individual wastewater is pumped into the interceptor, the effect is primarily one of blending together wastewaters having different concentrations of the various quality characteristics. Of the reactions that will be taking place among the various wastewater constituents, the most significant at the present time appears to be the combination of alkalinity and acidity. There will be a dampening of neutralization requirements as the alkaline wastewaters tend to neutralize the predominantly acidic wastewaters. Moreover, there are preliminary indications that the overall BOD load might be reduced slightly due to the interaction of all the wastewaters. The laboratory work to date indicates that the BOD of the integrated wastewaters is approximately 10 percent less than that calculated from the individual wastewaters. This, however, is based on a completely mixed system and the reduction in the interceptor would be considerably less.

The preliminary design and operating characteristics of the interceptor sewer permit the following conclusions to be drawn concerning equalization capacity in the interceptor:

1. The capacity provided in the individual equalization basins required for storm water runoff would have only a minor effect on equalizing quality characteristics.

2. The equalization capacity provided to level out fluctuations in the flow of process wastewater would have some effect on equalizing quality characteristics. Assuming typical basin design and an effective detention time of two days, variations in quality characteristics occurring over a 12-hour period would probably be effectively equalized.

3. There will be little or almost no opportunity for leveling out fluctuations in quality characteristics in the interceptor sewer because of its plug flow characteristics. There will be an opportunity for reactions to take place among the

constituents of the various wastewaters, but with the exception of neutralization this effect appears to be minor.

Equalization at the Regional Plant Site

In order to assay the need for providing additional equalization facilities at the regional treatment plant site, one must first consider those wastewater flow and quality characteristics which merit consideration in terms of regional plant equalization.

Flow:

Because the interceptor is being designed for partial length as a pressure system, it is not economically attractive to size facilities to handle peak flows. As previously discussed, tentative restrictions on fluctuations in the flow rate from each industrial source have been set at plus 10 percent of the design flow. Those industries pumping directly to the treatment plant would also be required not to exceed 10 percent of the design flow. These restrictions would reduce the need for surge basin requirements at the regional plant site.

Solids:

1. Suspended Solids - equalization is not required to dampen out fluctuation in the suspended solids load. Settleable solids will be removed in the primary clarifiers, and there is no real advantage in operating at a uniform concentration. Solids that can damage either the interceptor or treatment plant would not, however, be permitted in the system.

2. Dissolved Solids - biological processes are upset by large and rapid changes in the concentration of dissolved solids. The fluctuations must be substantial, however, and would have to exceed an increase of approximately 10,000 mg/l in less than 24 hours.

Biochemical Oxygen Demand:

Changes in the concentration of BOD do not usually upset activated sludge units unless the variation is large or a degree of toxicity is present. If the change results in a higher loading in terms of lbs BOD/lb MLVSS/day, the percent of BOD removal would decrease because activated sludge efficiency is responsive to loading.

The secondary clarification process following activated sludge can be upset if fluctuations in the BOD load result in sludge bulking. Although the cause of bulking is not fully understood, activated sludges have been difficult to contain under some loading conditions and particularly when the character of the BOD load is changing.

The data presented in Section IV indicate that the regional plant would normally have to be operated at fairly conservative activated sludge loadings to insure that effluent standards were met during periods of high BOD loads which "short-circuit" through the participant equalization basin and the regional interceptor.

Most of the variable BOD load can be attributed to a small volume of industrial waste flows such as DuPont Repauno. It would be much more economical to provide equalization basins for these flows rather than for the entire waste stream.

It should also be noted that the analytical data for the one industrial plant accounting for the high BOD load is based on several in-plant samples that are blended to give a representative sample. This method could result in more extreme variations in BOD concentrations than would occur if one representative stream were available for sampling. In any case, the need for equalizing the BOD load will depend on the situation at only a few of the participant industrial plants.

Neutralization:

Equalization of alkalinity and acidity is advantageous if there is a net savings in neutralization costs. Such a situation would occur if a waste stream varied from acidic to basic on an hourly or daily basis, but would tend to "self-neutralize" if there were sufficient detention time. There is no advantage, however, in equalizing a waste stream that is consistently acidic or basic because the net amount of chemicals required for neutralization remains essentially the same.

If one regional plant is constructed, the composite waste stream, according to the characterization data cited in Section IV, would always be acidic so there would be no advantage in equalization.

Potentially Toxic Constituents:

Materials that are capable of damaging the processes incorporated at the treatment plant, particularly the biological processes, will not be permitted in the regional system unless adequately diluted. Therefore, slug discharges of pesticides, solvents, large quantities of phenolic compounds, etc., will have to be regulated at the source by pre-treatment requirements.

Disinfection:

Bacterial analytical information has indicated that the industrial wastewaters are adequately disinfected by the low pH of the integrated waste stream. It is reasonable to assume that the municipal wastewaters would also experience some degree of disinfection if mixed with the industrial waters for a sufficient period of time. Therefore, there could be an advantage in equalization if it resulted in significant reduction in disinfection requirements.

Summary of Equalization

The need for equalization of the entire regional wastewater flow does not appear to be economically attractive or technically justified. This is underscored by the study conducted expressly for evaluating the effects of transient loadings using the Chambers Works flow. The results of this study are considered in Section VI of this report.

Neutralization

As part of the wastewater characterization program, the alkalinity, acidity, and amount of acid or base required to neutralize a sample to pH 7.0 were determined. In this task, these results were combined with flow data and analyzed with respect to each individual industry's location along with the proposed interceptor route to ascertain cumulative neutralization requirements.

Municipal wastewaters were not included in the neutralization calculations. Although domestic wastewaters typically have about 5.0 meq/l (250 mg/l $CaCO_3$) alkalinity, their pH values were usually in the range of 6.5 to 7.5 and therefore do not require neutralization.

Procedure

The amount of acid or base required to neutralize a sample to pH 7.0 was determined in accordance with Standard Methods (Reference 1). The results included herein are based on approximately 10 to 15 samples for each individual wastewater. Four analyses had been performed on the integrated wastewater and were available to check the cumulative requirements of the individual wastewaters.

The wastewater streams of two industries could not be sampled adequately before existing neutralization facilities. The neutralization requirements for these two wastewaters were therefore determined from plant operating records.

Results

1. The neutralization requirements for the individual wastewaters are presented in Table 7. The results are summarized in terms of high, low, and average requirements.

TABLE 7

NEUTRALIZATION REQUIREMENTS OF INDUSTRIAL WASTEWATERS

Wastewater	Condition	pH	Required to Neutralize to pH 7.0			
			Acid (a)		Base (a)	
			meq/l	Equiv/day	meq/l	Equiv/day
011	High	N.A.			28	15,000
	Low	N.A.			5	2,600
	Average	N. A.			16	8,500
021	High	9.1	2.18	44,000		
	Low	2.7			31.60	663,000
	Average	7.0±	-	-	-	-
031	High	8.2	0.44	40,000		
	Low	3.7			1.40	126,000
	Average	7.0±	-	-	-	-
041	High	3.5			18.50	2,700,000
	Low	2.0			2.46	360,000
	Average	2.6			8.88	1,300,000
061	High	2.5			121.50	1,370,000
	Low	1.2			5.70	64,000
	Average	1.8			74.0	840,000
071	High	11.4	5.56	6,100		
	Low	2.7			93.20	103,000
	Average	7.0±				
081	High	N. A.				
	Low	N.A.				
	Average	N.A.	0.56	6,300		
091	High	10.2	162.0	700,000		
	Low	1.3			104.0	450,000
	Average	2.0			100.0	430,000
101	High	11.0	5.0	5,000		
	Low	4.0			5.0	5,000
	Average	7.0±	-	-	-	-

(a) Equivalents/day based on 1975 flow. (preliminary estimate)
1 meq/l = 50 mg/l $CaCO_3$ - 37 mg/l $Ca(OH)_2$ = 40 mg/l NaOH

2. The cumulative neutralization requirements as the individual wastewaters were combined along the proposed interceptor route are summarized in Table 8. The cumulative requirements are presented for three different conditions: (1) typical effluent conditions at the individual plants, (2) the most basic conditions, and (3) the most acidic conditions.

3. The results for the typical conditions indicate that 8.7 meq/l of base would be required to neutralize the industrial waste stream at the regional treatment plant. For example, this amounts to 2,570,000 equivalents/day, or 176,000 lbs/day of 90 percent CaO at a flow of 78.6 MGD.

4. The most acidic conditions require approximately twice as much base for neutralization as do the typical conditions.

5. The most basic conditions indicate that a small amount of acid might be required to neutralize the industrial waste stream at the regional plant. The theoretical amount required, however, is quite small, and in view of the fact that unusual operating conditions would have to occur simultaneously at several plants, it is doubtful if this condition would ever occur.

6. Neutralization results for the integrated wastewater indicated that an average of 9.22 meq/l of base was required for neutralization. This compares favorably with the 8.7 meq/l figure previously cited.

Summary of Neutralization

The results of the neutralization studies indicate that the industrial wastewater stream at the Regional Treatment Plant would normally require approximately 8.7 meq/l of base for neutralization. The most acidic conditions experienced in the analytical program required approximately twice as much neutralization as the normal conditions. Because unusual operating conditions would have to occur simultaneously at several plants, it is doubtful if the pH of the wastewater stream would ever be above 7.0.

Chemical Coagulation and Flocculation

Studies were conducted on seven of the nine individual wastewaters to determine the potential for coagulation and flocculation as pretreatment. The two wastewaters that were excluded from the studies had been shown previously to have very little potential for pretreatment for suspended solids removal.

An integrated sample consisting of proportional volumes of the individual wastewaters was also analyzed to obtain a preliminary evaluation of its coagulation and flocculation potential before a more detailed evaluation was conducted during the operation of the pilot plant.

TABLE 8

CUMULATIVE NEUTRALIZATION REQUIREMENTS IN INTERCEPTOR

Interceptor Station	Flow	Typical Conditions Base Required		Most Basic Conditions Acid Required		Most Acidic Conditions Base Required	
	MGD	meq/l	Equiv/day	meq/l	Equiv/day	meq/l	Equiv/day
National Park	5.4	-	-	2.2	44,000	32.5	663,000
Mantua	8.7	(0.20)	(6,300)	1.7	56,400	23.1	759,700
Greenwich	34.0	3.4	432,200	6.2	793,800	7.8	1,004,700
Oldmans	37.0	3.1	432,200	6.0	843,800	7.5	1,054,700
Deepwater Treatment Plant	78.6	8.7	2,572,200	1.4	419,800	17.2	5,124,700

Procedure

The methodology was as follows:

1. Analyze raw waste sample for COD, suspended solids, pH and unusual characteristics.

2. Place one liter portions of raw waste in jars on a six-jar stirrer and check stirrer operation.

3. During a rapid mix of 100 rpm add the coagulant and mix for one minute. Use alum at doses of 2, 4, 8, 16, 32 and 64 mg/l.

4. Flocculate for 30 minutes at 30 rpm.

5. Settle for 30 minutes.

6. Visually observe the results. Measure the COD, suspended solids, and pH of the supernatant in the jar or jars which have the best visual results.

7. Repeat steps 1 and 2 for fresh samples of the raw waste.

8. Adjust pH of the one liter portions to 4, 5, 6, 7, 8 and 9 with sodium hydroxide or sulfuric acid.

9. To each jar add the optimum alum dose previously determined in steps 1 through 6.

10. Repeat step 6.

Results

The results for coagulation and flocculation without pH adjustment are summarized in Table 9. Table 10 summarizes results with pH adjustment. Wastewaters 061 and 091 had removals of over fifty percent in the chemical oxygen demand (COD), and subsequent tests were performed on these two wastewaters. These results are summarized in Table 11.

Wastewater 011:

When the wastewater was treated with a dose of 64 mg/l of alum, good flocculation occurred. At lower doses the particles were more discrete in nature, and little mechanical entrapment occurred. Good settling characteristics were found present with the 64 mg/l dose.

TABLE 9

SUMMARY OF RESULTS FOR FLOCCULATION WITHOUT pH ADJUSTMENT

Wastewater Sample Number	Optimum Alum Dose mg/l	Initial pH	Suspended Solids			Chemical Oxygen Demand			Comments
			Initial mg/l	Final mg/l	Percent Removal	Initial mg/l	Final mg/l	Percent Removal	
011	64	9.15	100	0	100	12,520	11,640	6.4	Large flocs formed.
041	8	2.02	20	20	0	373	365	2.1	No visual effect of alum.
061	32	1.78	100	40	60	1,059	863	18.5	Large number of floc particles.
071	8	1.79	320	20	93.8	74	58	21.6	Small flocs with good settling characteristics.
081	8	8.52	60	0	100	326	283	15.2	Some floating solids; clear supernatant.
091	8	1.38	6,500	90	98.9	17,600	8,400	52.3	Excellent settling; clear supernatant.
101	8	11.2	1,420	88	93.9	4,510	3,690	18.2	Slow settling but good solids removal.
191	8	3.0	0	0	--	573	585	1.4	No visible flocculation occurring.

TABLE 10

SUMMARY OF RESULTS FOR FLOCCULATION WITH pH ADJUSTMENT

Wastewater Sample Number	Optimum Alum Dose mg/l	Optimum pH	Suspended Solids[(a)] Initial mg/l	Suspended Solids[(a)] Final mg/l	Suspended Solids[(a)] Percent Removal	Chemical Oxygen Demand Initial mg/l	Chemical Oxygen Demand Final mg/l	Chemical Oxygen Demand Percent Removal	Comments
011	64	9.15	100	0	100.0	12,520	11,640	7.0	Good flocculation; clear supernatant.
041	8	9.10	(b) 20	(b) 20	(b) 0	419	376	10.3	Good flocculation at high pH.
061	32	7.01	660	60	91.0	1,120	556	50.3	Excellent flocculation. Particles come out of solution as pH is raised.
071	8	7.15	100	20	80.0	101	74	26.8	At lower pHs, poor settling; at high pHs excellent flocculation.
081	8	7.01	120	0	100.0	385	327	15.1	Clear supernatant; some floating solids.
091	8	7.00	6,500	60	99	17,600	18,400	-	Particles go into solution as pH is raised.
101	8	6.89	1,300	60	95.4	3,160	2,360	25.3	Particles in supernatant; slow settling.
191	8	9.30	(c) 0	(c) 0	(c) -	577	500	13.3	Good flocculation at high pH.

(a) Initial suspended solids refers to suspended solids concentration before pH adjustment.
(b) As pH is raised flocculant particles come out of solution.
(c) At pHs above 4.0, solids begin to come out of solution.

TABLE 11

RESULTS OF ADDITIONAL COAGULATION AND FLOCCULATION STUDIES

	WASTEWATER SAMPLE	
	061*	091**
Optimum Alum Dose mg/l	32	8
Optimum pH	3.09	1.46
COD		
Initial, mg/l	460	18,500
Final, mg/l	304	13,200
Percent Removal	33.9	28.6
BOD_5		
Initial, mg/l	260	9,575
Final, mg/l	226	7,950
Percent removal	13.1	17.0

*Good flocculent suspension

** Excellent settling

The effect of varying the initial pH of the wastewater sample was found to yield no additional removal of COD.

Although the optimum doses resulted in essentially the complete removal of the suspended solids, the reduction in COD was only 7.0 percent.

Wastewater 041:

Few solids were present in the raw wastewater and therefore little success was achieved by coagulation and flocculation.

As the pH of the wastewater was raised, however, a large volume of dissolved solids went into suspension. At high pH values, good flocculation was found to occur. Maximum removal in COD was found to be 10.3 percent.

Wastewater 061:

At the low pH of the raw wastewater sample, moderate success was achieved by the flocculation process. As the pH of the wastewater was raised, however, material came out of solution and excellent flocculating conditions developed. At an alum dose of 32 mg/l and a pH of 7.01, 90.0 and 50.3 percent removals were obtained for suspended solids and COD respectively.

Additional studies, including five-day BOD analyses, were performed on wastewater 061 and are summarized in Table 11. An alum dose of 32 mg/l at a pH of 8.09 produced a COD removal of 33.9 percent and a BOD removal of 13.1 percent. Although the subsequent test produced a smaller COD removal, the most significant fact is that the BOD removal is considerably less than that for COD. This would indicate that a large percentage of the suspended material can be chemically oxidized, but not biologically oxidized.

Wastewater 071:

Good flocculation and suspended solids removal were obtained for the wastewater both with and without pH adjustment. COD removals were not as good, however, with the maximum removal being 26.8 percent at a pH of 7.15 and an alum dose of 8 mg/l.

Wastewater 081:

The majority of solids contained in the sample were floating solids, and flocculation had no effect on them.

The fine suspended solids found present in the sample were found to flocculate well regardless of initial pH. COD and suspended solids removal were not found to be a function of initial pH. Maximum COD removal was 15.2 percent.

Wastewater 091:

Characteristically, this wastewater has a low pH and high suspended solids content. At a very low coagulant dose, high CODand suspended solids removals were achieved. While some bridging and flocculation occurred, the majority of the particles remained discrete. At an alum dose of 8 mg/l and a pH of 1.38, 98.9 and 52.3 percent removals were obtained for the suspended solids and COD respectively. While some removal can be attributed to coagulation and flocculation, most of the removal appeared to be the result of sedimentation.

No success was achieved by varying the initial pH of this wastewater because at higher pH values, the solids go into solution.

Subsequent studies resulted in a 28.6 percent removal of COD and a 17.0 percent removal of five-day BOD at an alum dose of 8 mg/l and an initial pH of 1.46. Although the COD removal was substantially less in this test, the BOD results indicate that the BOD load of this waste can be reduced significantly with a small amount of flocculation and settling.

Wastewater 101:

The raw wastewater sample contained a large number of particles for flocculation. At its raw pH, moderate removals were achieved.

The effect of varying the pH while keeping the dose constant was found to increase the removals slightly, but particles remained suspended in the supernatant. The optimum dose was 8 mg/l alum at pH of 6.89. The maximum reductions in suspended solids and COD was 95.4 and 25.3 percent respectively.

Wastewater 191 (Integrated Wastewater):

At its raw pH of 3.0, very few particles were present in the integrated wastewater sample and therefore flocculation resulted in negligible removals.

Characteristically, as the pH of the integrated sample is raised, dissolved material goes into suspension. Although the opportunity for flocculation improves at a higher pH values, the optimum dose and pH in this study resulted in a maximum COD removal of only 13.3 percent.

Summary

1. The results of this task did not indicate a significant potential for coagulation and flocculation as pretreatment for wastewaters 011, 041, 071, 081, and 101.

2. The results for wastewater 061 indicate that significant COD removals and smaller BOD removals can be achieved at a neutral pH with an alum dose of 32 mg/l. The high alum dose required, and the lesser effect on BOD tend to reduce the attractiveness of coagulation and flocculation on this wastewater.

3. The results for wastewater 091 indicate that significant COD, suspended solids, and BOD removals could be achieved with low alum doses at the acidic pH of the raw waste.

4. From the preliminary results for the integrated wastewater sample, it appears that there is not a significant potential for flocculation coagulation at the regional plant.

Effect of pH Adjustment without Chemical Addition

During the performance of the P-1 tasks, it became apparent that the integrated industrial wastewater had a considerable amount of dissolved material that tended to come out of solution as the pH was raised. In this study, the effect of pH adjustment as a sole method of treatment was further investigated, with particular attention given to the amount of base required for pH adjustment and the corresponding effect upon settleable solids and heavy metals.

Procedure

1. A titration curve of the integrated industrial wastewater was prepared using sodium hydroxide. The results were then plotted as pH versus meq/l of base added.

2. Four one liter samples of the integrated wastewater were placed in Imhoff Cones and the pH adjusted to approximately 7.0, 9.6 and 11.9 respectively. The pH of the fourth sample was not adjusted.

3. After one hour, the heavy metal concentration in the supernatant of each sample was measured.

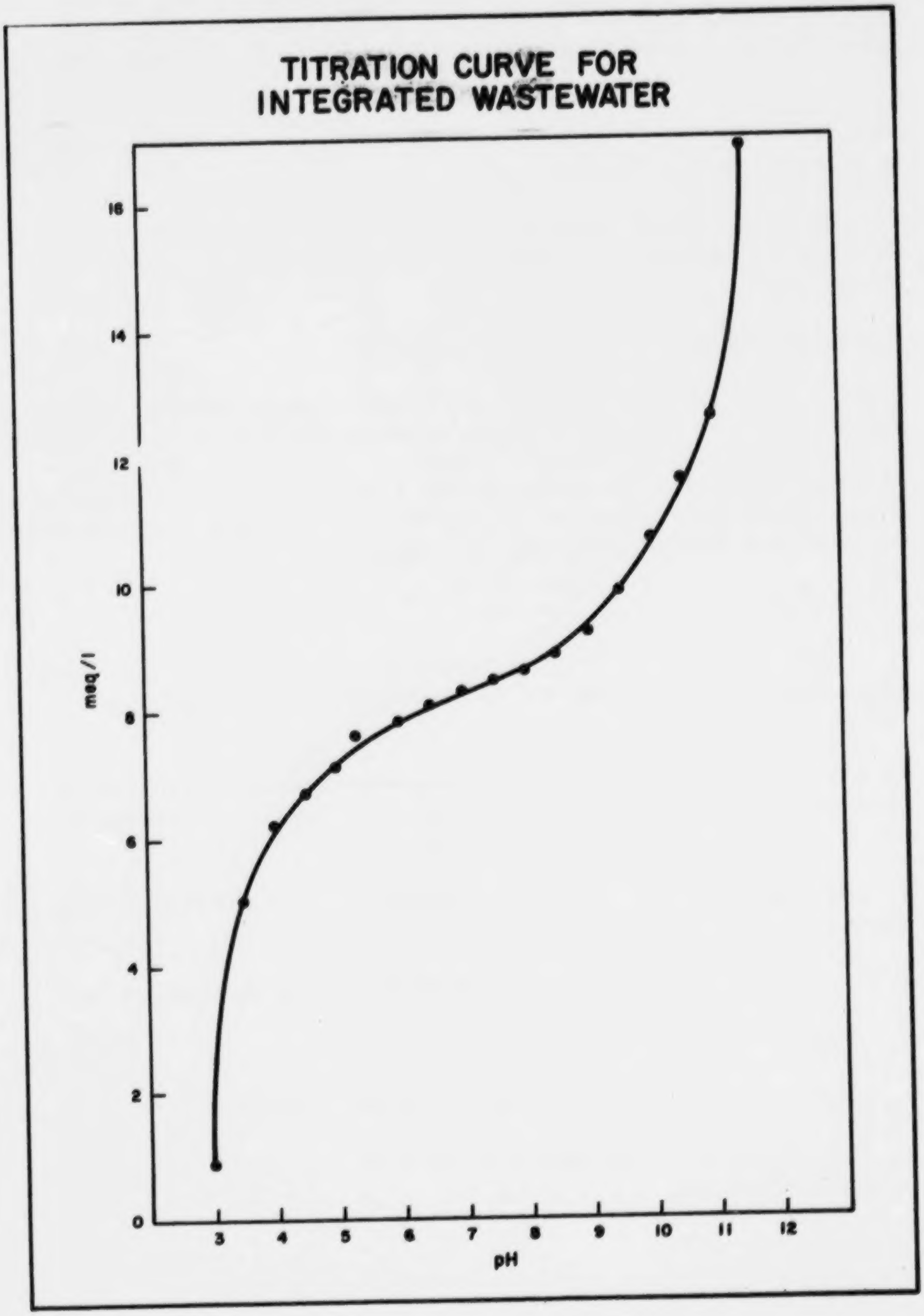
4. After 18 hours, the volume of solids in all samples was measured.

Results

1. The titration curve for the integrated wastewater is presented in Figure 5.

2. Table 12 summarizes the effect of pH adjustment on the solids present in the integrated wastewater.

Figure 5



TITRATION CURVE FOR INTEGRATED WASTEWATER

TABLE 12

SUMMARY OF EFFECT OF pH ADJUSTMENT ON SOLIDS IN THE INTEGRATED WASTEWATER

Sample	pH	Volume of Solids (a) ml	Observations
1	7.0	8.0	Fine particles, poor settling.
2	9.6	16.0	Fine particles, slow settling.
3	11.9	30.0	Particles very large, excellent settling, some color removal.
Unadjusted	3.2	0.01	Very few particles visible.

(a) After 18 hours settling in an Imhoff Cone.

3. Table 13 summarizes the effect of pH adjustment on heavy metals.

Summary

A large amount of settleable material can be removed from the integrated wastewater by raising the pH, with the effects becoming particularly significant above a pH of 7.1. At a pH of 11.9, large particles which settled rapidly were obtained. Seventeen meq/l (850 mg/l of $CaCO_3$) were required to adjust the pH to 11.9.

Similar effects were observed for heavy metal concentrations. With the exception of zinc and strontium, the heavy metals investigated were reduced below the 0.1 mg/l sensitivity of the spectrophotometer by adjusting the pH to 11.9.

Sedimentation Analyses of Untreated Wastewaters

Sedimentation analyses were conducted on the individual industrial wastewaters to determine the possible need for primary sedimentation at the individual plant sites. An integrated sample consisting of proportional volumes (based on 1975 flows) of the individual wastewaters was also analyzed in order to establish a preliminary evaluation for primary clarification efficiency at the future regional treatment plant.

Preliminary sedimentation analyses indicated that extensive analyses are not required at this time based on the low suspended solids concentrations of the individual wastewaters. The procedure as described herein was therefore used to delineate those streams potentially requiring gravity separation from those where it was not deemed necessary.

Procedure

The methodology was as follows:

1. Each sample was neutralized to a pH of 7.0 and thoroughly mixed. A volume of one hundred ml was then withdrawn for an initial suspended solids analysis.

2. One liter of the neutralized sample was then placed in a 1000 ml graduated cylinder equipped with sampling ports.

3. After a settling time of 10 minutes, 100 ml was removed from the sample port located 11.2 inches below the initial water surface in the cylinder. This sample was then analyzed for a final suspended solid concentration.

The settling that occurs under these conditions is indicative of that which would occur in a clarifier with an overflow rate of approximately 1000 gpd/ft^2.

TABLE 13

SUMMARY OF THE EFFECT OF pH UPON HEAVY METALS IN THE INTEGRATED WASTEWATER

pH	Cr, mg/l	Cu, mg/l	Fe, mg/l	Ni, mg/l	Pb, mg/l	Zn mg/l	Mn mg/l	Ag, mg/l	Sr, mg/l
11.90	<0.1	<0.1	<0.1	<0.1	<0.1	0.8	<0.1	<0.1	0.4
9.60	0.2	0.2	0.2	<0.1	<0.1	0.3	<0.1	<0.1	0.3
7.11	0.2	0.2	0.3	<0.1	<0.1	0.7	0.4	<0.1	0.4
3.20	0.6	0.7	8.0	<0.1	0.5	0.8	0.5	<0.1	0.4

All samples that were investigated were collected from the individual plants at points above any gravity separation facilities.

Results

All results are summarized in Table 14. Only the wastewater designated 101 demonstrated a potential for requiring sedimentation. The remaining individual wastewaters were sufficiently low in suspended solids concentrations and it would not appear feasible to require sedimentation as pretreatment at these plant sites.

Almost all of the solids that were removed from wastewater 081 floated readily to the surface, thus indicating a potential of flotation as pretreatment.

Two samples had sufficient quantities of floating oil to indicate the need for in-plant control.

The integrated wastewater had an initial suspended solids concentration of 130 mg/l with a removal of 23 percent under the aforementioned settling conditions. Two significant factors were apparent based on these bench scale studies, namely, the solids were of a flocculent nature, and the concentrations appeared to be pH dependent.

BIOLOGICAL TREATMENT EVALUATION

Secondary biological treatment is applied to reduce the concentration of organic wastewater constituents through biochemical oxidation to a level acceptable for discharge into a receiving body of water or to the point where tertiary treatment can be employed effectively. Although the applicability of biological processes for domestic wastewater treatment is well documented, bench or pilot scale biological treatability tests should be conducted where industrial wastewaters are involved. Such testing programs yield data which are necessary in predicting the levels of effluent quality which can be obtained and the design factors required to achieve these effluent quality goals.

The scope of the biological treatment evaluation using the bench scale approach as originally proposed included only the use of batch reactors. However, it was assumed that a more representative simulation study would be required in order to accurately define the response of each individual wastewater to biological treatment. Consequently, the scope was expanded to include the evaluation of biological treatment for each industrial wastewater and the integrated composite using continuous bench scale reactors. These studies were conducted over a period of three months.

The general procedure for the treatability studies involved operating one continuous

TABLE 14

SEDIMENTATION ANALYSES OF UNTREATED WASTEWATER

WASTEWATER	Suspended Solids Initial mg/l	Suspended Solids Final mg/l	Percent Removal	OBSERVATIONS
011	180	180	0	No visible solids.
021	70	50	28.6	Large amount of floating oil. No solids visible.
031	80	70	12.5	Solids are discrete, very fine. Some floating oil present.
041	130	50	61.6	Very few particles present. Appear discrete.
051				No samples available.
061	110	80	27.3	Discrete particles, very few visible.
071	170	70	58.9	Large fragile flocs.
081	210	60	71.4	Most of solids floated to surface.
091	170	90	47.0	Many solids appeared to go into solution when pH adjusted from 1.1 to 7.0. Remaining particles are discrete.
101	750	120	84.0	Large discrete particles. Rapid settling.
191	130	100	23.0	Solids became visible when pH adjusted from 2.6 to 7.0. Small flocs visible after 10 minutes. After 30 minutes large non-settling flocs visible.

reactor for each industrial wastewater including an integrated sample made up of proportional volumes of the individual wastewaters. Each unit was evaluated at three different organic loading rates for approximately three weeks or until sufficient characterization data had been obtained at each loading condition.

Twenty-four hour composite wastewater samples were collected at the individual industrial plants three times per week as described in Section IV. One gallon of each sample was split off for use in the wastewater characterization program, and the remaining volume was stored for use as feed for the biooxidation units. Typically, the individual samples for each wastewater were accumulated for one to two weeks in a 50 gallon drum, with each drum maintained at a pH of 2.0 or less to prevent bacterial decay. This accumulated sample was then used as feed to the biological reactors.

Acclimation of the Biological Seed

Prior to the operation of the continuous biooxidation units, activated sludge organisms were acclimated to the individual wastewaters. The units used for acclimation consisted of a four-liter Erlenmeyer flask kept under a small vacuum. The acclimated cultures were aerated by drawing prefiltered air through the cultures. The air suction line also served as a constant level control and sludge removal line. Excess cells were collected in a second Erlenmeyer flask which acted as a liquid trap between the acclimation flask and main vacuum line. Initially, the cultures were fed manually. However, after tests indicated that viable cultures had developed, the cultures were fed continuously by means of a Dekastaltic pump.

Several sources of seed were used to develop the acclimated organisms, including domestic activated sludge from the City of Wilmington, the activated sludge treatment plant operated by Hercules, Incorporated in Gibbstown, New Jersey, and acclimated seeds maintained at the Wastewater Laboratory duPont, Chambers Works, Deepwater, New Jersey.

Operation of Acclimation Units

Initial loading of the individual units were based upon the available information about the individual waste streams and in each case, the seed sludge was selected from that source which was most like the corresponding waste. During the first days each seed culture was examined microscopically at least twice a day and frequent adjustments were made in the rate and dilution of the waste used as feed. After one week all cultures had stabilized, and a regular feed program was initiated. Determinations of volatile suspended solids and oxygen uptake were made during the acclimation period to ascertain that the acclimated seeds remained active. A summary of the results for the individual acclimated seeds are presented in

Table 15. The results show that all seeds were active at the completion of the acclimation period.

Experimental Biological Reactors

The experimental apparatus used on each wastewater consisted of a continuous reactor, a feed pump, feed and effluent bottles, and an air supply. The primary element of each system was the biooxidation unit, a schematic of which is shown in Figure 6. Ten of these units were obtained from BioDevelopment Associates, Austin, Texas. Each unit has an aeration chamber with a maximum capacity of eight liters, a two liter clarification chamber, and an adjustable overflow weir for control of the working volume. The aeration chambers have completely mixed flow characteristics, and settled solids from the clarification chamber are recycled by induced hydraulic action. Air from a central supply system was bubbled through a stone diffuser to provide dissolved oxygen and provide mixing for the individual units.

The wastewater feed system consisted of individual feed bottles and one central Dekastaltic pump with ten channels. Each pumping channel consists of a Tygon tube looped around a central variable speed rotor with three roller bars. Flow variations can be achieved by varying the tubing size and the motor speed. The complete biological reactor system as set up in the laboratory is shown in Figure 7.

Operating Procedures

The following basic procedures were generally followed during the treatability studies:

(1) The previously mentioned industrial wastewater samples were accumulated in 50 gallon storage drums, one for each individual wastewater, for one to two weeks.

(2) When the necessary volume had accumulated, proportional samples were taken from each individual drum and mixed to give an integrated wastewater that was representative of the industrial wastestream that would be treated at the proposed regional treatment plant. The percentage of each individual wastewater used for the integrated wastewater was cited previously in Section IV.

(3) The feed stock in all the storage drums was analyzed for total suspended solids (TSS), volatile suspended solids (VSS), five-day biochemical oxygen demand (BOD_5), chemical oxygen demand (COD), total organic carbon (TOC), total Kjeldahl nitrogen (TKN), nitrite and nitrate nitrogen ($NO_2 + NO_3$), total phosphorus, phenols, and methylene blue active substances (MBAS).

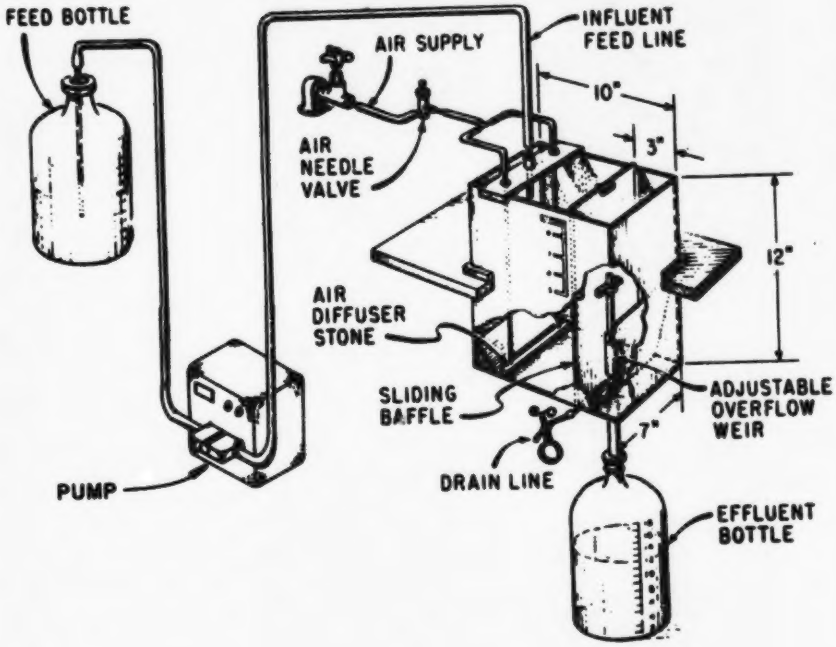
TABLE 15

ACTIVITY OF ACCLIMATED SEEDS
DISSOLVED OXYGEN UPTAKE

Tabulated values are mg dissolved oxygen per liter

	WASTEWATER CODE										
Time (Sec.)	10	20	30	40	50	60	70	80	90	100	200[(a)]
0	7.0	8.1	8.1[(d)]	6.2	(c)	1.2	7.6	7.5	1.5	7.0	8.0
15	6.6	8.0	8.0	4.8		0.9	7.5	7.4	1.5	6.8	7.9
30	6.2	7.8	7.8	3.0		0.7	7.4	7.3	1.5	6.7	7.8
60	5.8	7.6	7.6	2.7		0.7	7.4	7.2	1.4	6.4	7.8
180	4.5	7.4	7.4	1.6		0.5	7.2	6.8	0.5	5.7	7.5
300	3.5	7.0	7.0	0.9		0.4	6.9	6.3	0.4	5.1	7.2
420	2.7	6.6	6.6	0.4		0.2	6.7	5.9	0.2	4.4	6.8
540	1.8	6.2	6.2	0.2		0.2	6.3	5.4	0.2	3.8	6.5
600	1.4	6.0	6.0	0.1		0.2	6.2	5.1	0.2	3.5	6.3
900	--	5.1	5.1	0.0		0.1	5.5	3.8	0.1	-	5.5
VSS[(b)] (mg/l)	3,200	1,780	1,320	1,820		1,800	2,080	2,200	1,800	2,400	1,520

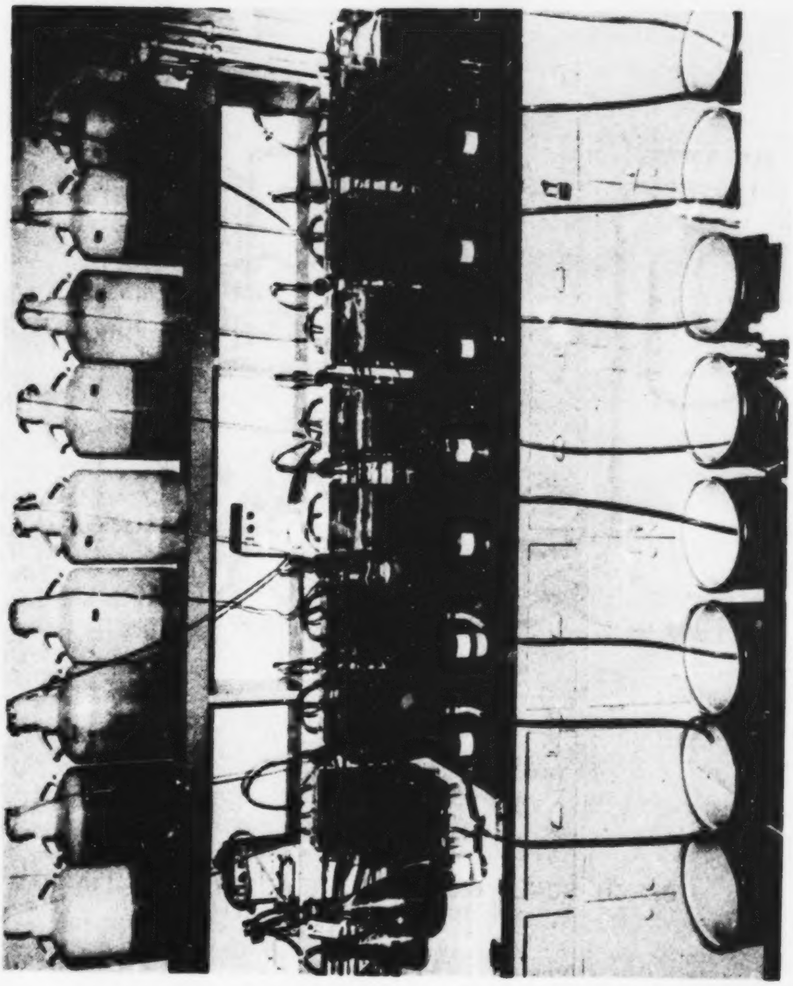
(a) Composite of all wastes.
(b) Mixed liquor volatile suspended solids.
(c) No waste available.
(d) Same seed used for 20 and 30.


FEED BOTTLE
AIR SUPPLY
INFLUENT FEED LINE
10"
3"
AIR NEEDLE VALVE
12"
AIR DIFFUSER STONE
SLIDING BAFFLE
ADJUSTABLE OVERFLOW WEIR
7"
DRAIN LINE
PUMP
EFFLUENT BOTTLE

BENCH SCALE BIOLOGICAL REACTOR FLOW DIAGRAM

Figure 7

BENCH SCALE BIOLOGICAL REACTORS



(4) Each day sufficient amounts of each wastewater were removed from the individual storage drums to serve as feed to the corresponding biooxidation unit for the following 24 hour period. This sample was neutralized and, if necesary, nutrients added. In some cases, dilution of the feed stock was necessary to permit adequate control of the feeding rate. The feeding rate was measured daily.

(5) Each day the following tests were conducted on the mixed liquor of each unit: suspended solids, volatile suspended solids, oxygen uptake, pH, and temperature.

(6) The effluents from each unit were collected in containers and analyzed according to the following schedule:

- (a) Once a day an effluent sample was filtered and analyzed for COD and TOC.
- (b) Twice a week the BOD_5 of the filtered sample was measured.
- (c) Once a week the filtered effluent was analyzed for TKN, $NO_2 + NO_3$, total phosphorus, phenol, and MBAS.
- (d) The carbonaceous oxygen demand of the filtered effluent was determined once for each loading condition.
- (e) Once a week the COD of settled effluent was determined.

The above procedures were modified at times according to the response of the individual units.

Theory of Biological Treatment

When evaluating the biological treatability of wastewaters, it is important to consider the constituents which adversely affect the performance and capacity of the system. This is particularly true when developing design information from bench or pilot scale studies. Although the limiting or inhibitory threshold concentrations of specific constituents on biological performance fluctuate, approximate values are reported in Table 16. Once those constituents which may affect biological treatment are defined, continuous-flow and batch biological reactor systems can be used in the laboratory to assess the treatability and predict the process kinetics. Most pilot plant operations, however, are continuous-flow systems. The batch analysis approach is usually limited to screening tests, seed acclimation, and generalized estimates of organic removals, as the continuous-flow process analyses provide a more accurate basis for predicting process kinetics and establishing design criteria.

It is desirable to relate the biological oxidation system to a mathematical model,

TABLE 16

PRE-OR PRIMARY TREATMENT REQUIREMENTS

Constituent	Limiting or Inhibitory Concentration	Treatment
Suspended Solids	>125 mg/l	Lagooning, sedimentation, flotation
Oil or Grease	>100 mg/l	Skimming tank or separator
Heavy Metals	<1-10 mg/l	Precipitation or ion exchange
Alkalinity	0.5 lbs alkalinity as $CaCO_3$ per lb BOD removed	Neutralization for excessive alkalinity
Acidity	Free mineral acidity	Neutralization
Organic load variation	>4:1	Equalization
Sulfides	>100 mg/l	Precipitation or stripping
Chlorides	>8,000-25,000 mg/l	Dilution, deionization
Phenols	>70-160 mg/l	Stripping, provide complete mixing
Ammonia	>1,600 mg/l	Dilution; pH adjustment and stripping
Dissolved salts	>16,000 mg/l	Dilution, ion exchange

determining the coefficients from bench or pilot scale studies. This includes an evaluation of substrate removal, sludge production, and oxygen requirements.

There is an increasing use of completely mixed biological systems, particularly in the activated sludge treatment of industrial wastes. In this case, the soluble BOD in the effluent is equal to that in the aeration tank. A material balance results in the following relationship:

$$Q\ S_o - Q\ S_e = \frac{dS}{dt}\ .V \qquad \text{(V-l)}$$

where:

S_o = raw waste COD, BOD

V = tank volume

S_e = effluent COD, BOD

t = detention time

Q = flow

Substituting the simplest form of $\frac{dS}{dt}$ in terms of a retardent equation will yield the relationship:

$$\frac{S_o - S_e}{X_a t} = K S_e^{\ n} \qquad \text{(V-2)}$$

where:

X_a = VSS undergoing aeration
K = substrate removal rate
n = exponent (for a first order approximation, n=l)

The total oxygen requirements in a biological system are related to the oxygen consumed to supply energy for synthesis and the oxygen consumed for endogenous respiration. This assumes that oxygen must be supplied to the system in order to:

(1) provide oxygen for biological organic removal ($a'S_rQ$),

(2) provide oxygen for endogenous respiration where cells lyse and release soluble oxidizable organic compounds ($b'X_aV$), and

(3) provide oxygen required for chemical oxidation as measured by the immediate oxygen demand (k^oQ).

This expression is:

$$R_rV = a'S_rQ + b'X_aV + k^oQ \qquad (V-3)$$

where:

R_r = oxygen utilization per day

V = volume of aeration basin

a' = fraction of substrate (BOD or COD) used for oxidation

S_r = substrate (BOD and COD) removed

Q = flow

b' = fraction per day of VSS oxidized (oxygen basis)

X_a = av MLVSS in aeration tank

k^o = chemical oxygen demand coefficient (as measured by immediate oxygen demand)

Sludge accumulation in the activated sludge system from the biological oxidation of wastewaters can be computed using a similar approach. The components of a mathematical relationship would include:

(1) increase in sludge attributable to influent SS (QX_i)

(2) increase in sludge due to cellular sythesis (aS_rQ)

(3) decrease in sludge due to cellular oxidation or endogenous respiration (bX_aV)

(4) decrease in sludge due to effluent SS (QX_e)

The expression is:

$$\Delta X = [QX_i + aS_rQ] - [bX_aV + QX_e] \qquad (V-4)$$

where:

ΔX = sludge production per day

V = volume of aeration basin

Q = flow

a = fraction of substrate (COD, BOD) converted to new cells

S_r = substrate (BOD or COD) removal

b = fraction per day of VSS oxidized (sludge basin)

X_a = average MLVSS in aeration tank

X_i = influent SS

Xe = effluent SS

A graphical solution for determining the design coefficients can be obtained by varying organic loadings to the bench or pilot units and measuring the parametric responses. The substrate removal rate from Equation (V-2) can be estimated by plotting the response data in accordance with Figure 8(A). If a non-removable COD or BOD persists as shown in Figure 8(B), then Equation (V-2) must be modified accordingly:

$$\frac{S_o - S_e}{X_a t} = KS_e - y \qquad \text{(V-5)}$$

The system oxygen requirements can be estimated by rearranging Equation (V-3):

$$\frac{R_r}{X_a} = \frac{a'S_r}{X_a t} + b' \qquad \text{(V-6)}$$

where $t = \frac{V}{Q}$, and $k\,^{o}Q$ is neglected assuming this oxygen demand is satisfied prior to testing. The a' coefficient is taken as the slope and b' as the intercept when plotting the data as shown in Figure 9(A).

The synthesis sludge production is predicted by rearranging Equation (V-4) and neglecting or accounting for the influent and effluent suspended solids:

$$\frac{\Delta X}{X_a} = \frac{aS_r}{X_a t} - b \qquad \text{(V-7)}$$

the "a" and "b" coefficients are taken as the slope and intercept values, respectively, of the plot shown in Figure 9(B).

Figure 8

SUBSTRATE REMOVAL RATE

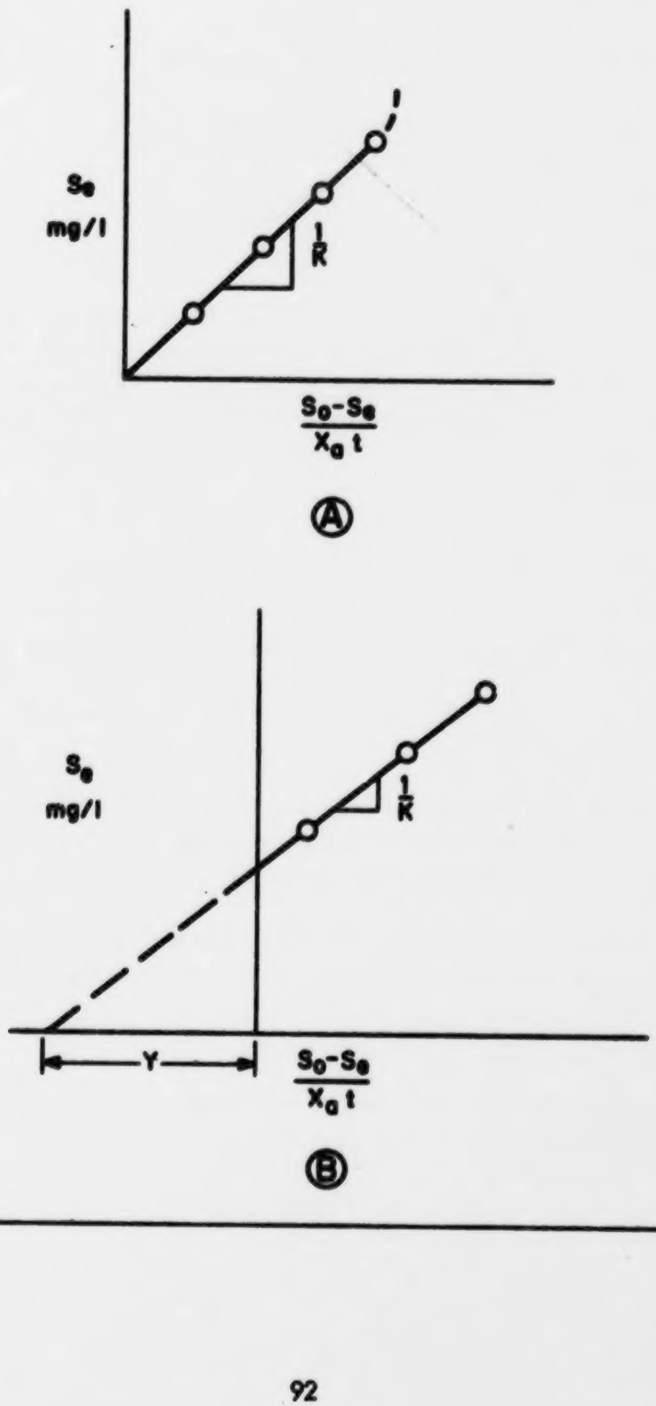
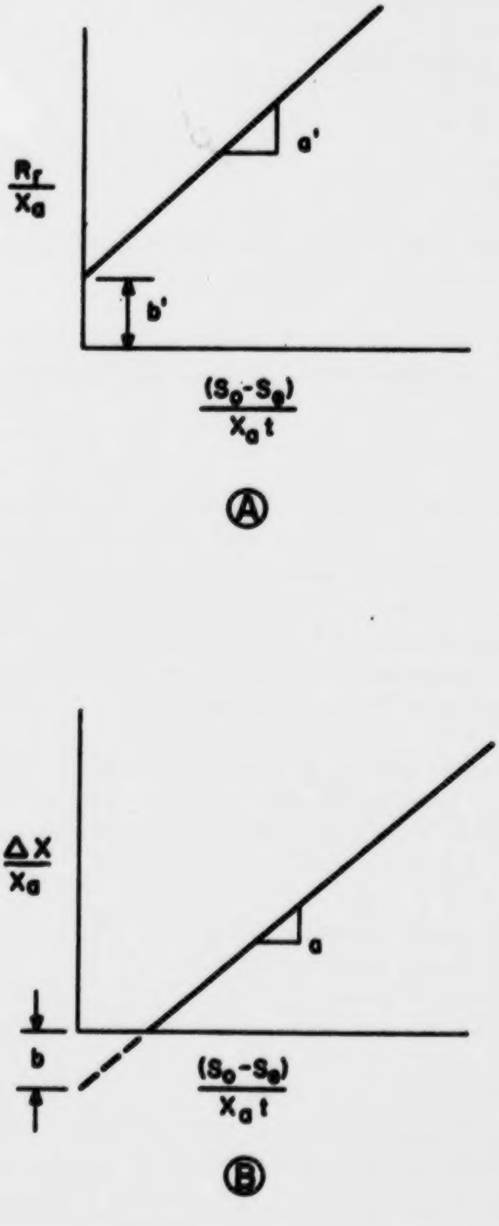


Figure 9

OXYGEN REQUIREMENTS AND SLUDGE PRODUCTION



It is to be emphasized that a key parameter in the analysis of the data is:

$$\frac{S_o - S_e}{X_a t} \qquad \text{(V-8)}$$

This parameter hereafter will be referred to as the removal velocity and has the units pounds substrate removed/pound MLVSS/day.

An equally important parameter is:

$$\frac{S_o}{X_a t} \qquad \text{(V-9)}$$

This parameter hereafter referred to as the organic loading and has the units of pounds substrate applied/pound MLVSS/day.

It should be noted that the removal velocity is approximately equal to the load when the effluent concentration of the substrate (S_e) is small.

Data Management

Because of the considerable amount of data that was generated during the course of the treatability studies, it was essential that efficient data handling methods be utilized from the start. The procedures were as follows: (1) basic analytical results were recorded on typical laboratory data sheets; (2) these data were then transferred to a standard data sheet that could be read by a key punch operator; (3) the data were then punched on computer cards; and (4) the data were read into an IBM 360 computer and processed by a Fortran IV program.

While the studies were in progress, a simplified computer program was incorporated for monitoring results. After the completion of the studies, the program was expanded so that the output for each individual wastewater consisted of seven sheets as follows:

(1) a summary of results based on BOD_5
(2) a summary of results based on COD
(3) a summary of results based on TOC
(4) a summary sheet for organic removals in terms of BOD_5, COD, TOC, phenols, and MBAS
(5) a summary of influent conditions
(6) a summary of filtered effluent conditions, and
(7) a summary of the mixed liquor conditions.

The computer program is outlined in Table 17.

TABLE 17

COMPUTER PROGRAM FOR TREATABILITY STUDIES

```
IV G LEVEL 1, MOD 4                 MAIN          DATE = 70120      07/26/38

      DIMENSION  X(16),  X3(20, 16), X5(50,5),  X6(20,3),  X7(50,3),
     1 X4(50,16), ID3(20), IM3(20), IY3(20),ID5(50), IY5(50), IM5(50), ID6(20),
     21M6(20), IY6(20), ID7(50), IM7(50), IY7(50), ID4(50), IM4(50), IY4(50),
     3DIL(50), BODIN(50), FLO(50), T(50), CIN(50), VSS(50), SA(50),
     4SLUDGE(50), DT(50), PCTB(50), PCTC(50), PCTT(50), PHENL(50), SMBA(50),
     5PCTP(50), PCTM(50), TOCIN(50), CODS(50)
      CODSET=1000000.
    2 N3=0
      I3=0
      N4=0
       XFOUR=1.0E30
      XFIVE=1.0E-10
      I4=0
      N5=0
      I5=0
      N6=0
      I6=0
      N7=0
      I7=0
    1 READ(1,100) IS,              IYR, IMO, IDAY,       (X(J),J=1,16),V
      ISAM=IS-   3
      IF(IS  ) 10,10,20
    3 READ(1,101)  IS1,IS2, IS3, IYR, IMO, IDAY, (X(J), J=1, 16), V1
      IF(V1.NE.0.) V=V1
      IF(IS1) 11,11,12
   12 IF(IS3.NE.3)  GO TO 25
   20 N3=N3+1
      I3=I3+1
      IM3(I3)  =IMO
      ID3(I3)  =IDAY
      IY3(I3)  =IYR
      IF(X(1). NE.0.0)FLOW=X(1)
      DO 21  J=1,16
      X3(I3,J)=X(J)
      IF(X3(I3,J))22,22,21
   22 X3(I3,J) =1.0E20
   21 CONTINUE
      GO TO 3
```

TABLE 17 (continued)

```
 25   IF(IS3.NE.5)  GO TO 30
      N5=N5+1
      I5=I5+1
      IM5(I5)=IMO
      ID5(I5)=IDAY
      IY5(I5)=IYR
      X5(I5,1)= X(2)
      X5(I5,2)= X(3)
      X5 (I5,3)= X(4)
      X5(I5,4)= X(5)
      IF(X(5).NE.0.) XFIVE=X(5)
      X5(I5,5)=X(10)
      DO  26  J=1,5
      IF(X5(I5,J))27,27,26
 27   X5(I5,J)=1.0E20
 26   CONTINUE
      GO TO 3
 30   IF(IS3.NE.6)  GO TO 35
      CODSET=X(8)
      IF(CODSET.EQ.0.0)CODSET=1.0E 10
 31   GO TO 3
 35   IF(IS3.NE.7)  GO TO 40
      N7=N7+1
      I7=I7+1
      X7(I7,1)  =X(1)
      X7(I7,2)  = X(4)
      X7(I7,3)  = X(5)
      DO   36    J=2,3
      IF (X7(I7,J))37,37,36
 37   X7(I7,J)=1.0E10
 36   CONTINUE
      GO TO 3
 40   N4=N4+1
      I4=I4+1
      PHENL(N4) = X3(I3,15)
      BODIN(N4)= X3(I3,6)
      DIL(N4)=X3(I3,11)
      FLO(N4)=FLOW
      VOL(N4)=V
      TOCIN(N4)=X3(I3,9)
      CIN(N4)=X3(I3,8)
      CODS(N4)=CODSET
      VSS(N4)=XFIVE
      SMBA(N4)=X3(I3,16)
```

TABLE 17(continued)

```
      IM4(I4)  = IMO
      IY4(I4)  =IYR
      ID4(I4)  =IDAY
        IF(X(6).NE.0.)XFOUR=X(6)
      DO 41 J=6,16
      X4(I4,J)=X(J)
      IF(X4(I4,J))42,42,41
   42 X4(I4,J)=1.0E20
   41 CONTINUE
          X4(I4,4)=0.
        X4(I4,5)=0.
      IF(I4.NE.1)  GO TO 43
      WRITE(3,200) I SAM
      WRITE(3,205)
      WRITE (3,202)
   43 DT(N4)=VOL(N4)/FLO(N4)
      ALOAD  = (FLOW     *    X3(N3,6)   /  (V*  XFIVE)
      REMV    =(FLOW      *  (X3(N3,6) - XFOUR)    )  /  (V*  XFIVE)
      PCTB(N4)   =100.*(X3(N3,6)-X4(N4,6))/X3(N3,6)
      SLUDGE(N4)  =   X7(N7,1) *X7(N7,3) /1000.
      SA(N4)   =(X7(N7,1)   * X7(N7,3))   /(V*XFIVE)
        IF(X7(N7,1).EQ.0.) SA(N4)=1.0E10
      IF(X4(N4,6).LT.1.0E05)GOT044
      PCTB(N4)=1000000.
      CODSET=1000000.
      GOT03
   44 WRITE(3,201)  IMO,IDAY,IYR,V,X3(N3,1), DT(N4), X3(N3,6), X4(N4,6),
     1ALOAD,REMV, PCTB(N4), X4(N4,7), SLUDGE(N4), SA(N4),VSS(N4)
      CODSET=1000000.
      GO TO 3
   11 WRITE(3,271)ISAM
      WRITE(3,207)
      WRITE(3,202)
      DO  71  I=1,N4
      ALOAD=(FLO(I)   * CIN(I))/(VOL(I)   * VSS (I))
      REMV  = (FLO(I)   * (CIN(I) -X4(I,8))/(VOL(I)*VSS(I)     )
      PCTC(I)=100. *(CIN(I)-X4(I,8))/C IN(I)
      IF(CIN(I).LT.1.0E05)GOT072
      GOT071
   72 IF(X4(I,8).LT.1.0E05)GOT070
      PCTC(I)=1000000.
      GOT071
   70 WRITE(3,270)IM4(I), ID4(I), IY4(I),VOL(I),FLO(I),DT(I),CIN(I),
```

TABLE 17(continued)

```
     1X4(I,8),ALOAD,REVM,PCTC(I),CODS(I), SLUDGE (I),SA(I) ,VSS(I)
  71   CONTINUE
 206   FORMAT(14X,'L',8X,'L/DAY     DAYS',6X,'MG/L',6X,'MG/L',46X,
     1'G/DAY    G/G*DAY     MG/L')
       WRITE(3,281)  ISAM
       WRITE(3,206)
       WRITE(3,202)
       DO 81  I=1,N4
       PCTT(I)=100. *(TOCIN(I)-X4(I,9))/TOCIN(I)
       ALOAD=( FLO(I)*TOCIN(I))/(VOL(I)*VSS(I))
       REMV=(FLO(I) *(TOCIN(I)-X4(I,9)))/(VOL(I)*VS S(I)        )
       IF(TOCIN(I).LT.1.0E05)GOT082
       REMV=1000000.
       PCTT(I)=1000000.
       GOT081
  82   IF(X4(I,9).GT.1.0E05)GOT081
  80   WRITE(3,280)  IM4(I),ID4( I),IY4(I),VOL(I),FLO(I),DT(I),TOCIN(I),
     1 X4(I,9),ALOAD,REMV,PCTT(I), SLUDGE(I),SA(I),VSS(I)
  81   CONTINUE
       WRITE(3,291) ISAM
       DO 90 I=1,N4
       PCTP(I)= ICO.*(PHENL(I)-X4(I, 15))/PHENL(I)
       IF(PHENL(I).GT.1.0E05)PCTP(I)=1000000.
       PCTM(I)=100.*(SMBA(I) -X4(I, 16))/SMBA(I)
       IF(SMBA(I).GT.1.0E05)PCTM(I)=1000000.
  90   WRITE(3,290) IM4(I),ID4(I),IY4(I),DIL(I),BODIN(I),CIN(I), TOCIN(I).
     1 PHENL (I) SMBA (I), X4 (I,6), (X4(I,J), J=8,9), X4(I,15), X4(I,16),
     2 PCTB(I),PCTC(I),PCTT(I),PCTP(I),PCTM(I)
 290    FORMAT(1X,I2,2X,I2,1X,I2,5X,F3,0,5X, 3(F5.0.1X),F7.3,1X,F4.1.
     16X,3(F5.0, 1X),F7.3,1X,F4.1X,F4.1,6X,4(F5.1,1X),1X,F5.1)
 291    FORMAT(1H1 , 'TREATABILITY STUDY FOR WASTEWATER ',I3/
     1'  SUMMARY OF ORGANIC REMOVALS'/
     270X,'FILTERED'/
     312X,'DILUTION      INFLUENT  CONCENTRATIONS',12X,'EFFLUENT',
     42X 'CONCENTRATIONS',15X,'PERCENT  REMOVALS'/15X'OF'/
     5' MO DAY YR RAW WASTE   BOD    COD    TOC PHENOL  MBAS',7X ,
     6'BOD  COD  TOC  PHENOL  MBAS',7X,'BOD  COD  TOC PHENOL  MBAS'/
     711X,'WATER/WASTE  MG/L  MG/L  MG/L  MG/L   MG/L'
     87X,'MG/L  MG/L  MG/L   MG/L'//)
 205    FORMAT(14X,'L',8X,'L/DAY  DAYS',6X,'MG/L,6X,'MG/L',36X,
     1'MG/L',6X,'G/DAY   G/G*DAY    MG/L')
 207     FORMAT(14X,'L',8X,'L/DAY         DAYS',6X,'MG/L',6X,'MG/L',23X,
     1'(FILTERED)',3X,'MG/L',6X,'G/DAY  G/G*DAY          MG/L')
       WRITE(3,230)ISAM
```

TABLE 17(continued)

```
      WRITE(3,231)
      DO  53  I3 = 1,N3
53    WRITE(3,224)  IM3(I3), ID3 (I3), IY3(I3),
     1X3(I3,6),
     2X3(I3,8),X3(I3,9),
     3(X3(I3,J),J=12,16),X3(I3,11)
      WRITE(3,223)   ISAM
      WRITE(3,232)
      DO  60  I4=1,N4
60    WRITE(3,229)  IM4(I4),ID4(I4),IY4(I4),X4(I4,6),
     1X4(I4,8),X4(I4,9),
     2 (X4(I4,J),J=12,16)
        WRITE(3,240) ISAM
      WRITE(3,225)
      DO  54  I5=1,N5
54    WRITE (3,226)IM5(I5),ID5(I5),IY 5(I5),
     1(X5(I5,J),J=1,5)
      GO TO 2
100   FORMAT(I3,I2,I2,I2,F5.3,F3.1,F2.0,6F5.0,F3.2,F2.0,
     13F4.1,F6.3,F3.1,F3.2)
101     FORMAT(3I1,3I2,F5.3,F3.1,F2.0,6F5.0,F3.2,F2.0,3F4.1,F6.3,F3.1,
     1F3.2)
200     FORMAT(1H1,'TREATABILITY STUDY FOR WASTEWATER',I3/
     1' SUMMARY OF RESULTS BASED ON BOD'/
     2' LOADING AND REMOVAL VELOCITY ARE EXPRESSED AS LBS BOD5 / LBS  MLV
     3SS  * DAY'//
     451X,'FILTERED' , 23X,'PERCENT  ULTIMATE'/
     511X,  'VOLUME', 13X, 'DETENTION  INFLUENT  EFFLUENT',
     613X, 'REMOVAL  REMOVAL  EFFLUENT  SLUDGE  GROWTH'/
     7' MO  DAY  YR  OF UNIT  FLOW  RATE   TIME    BOD5    BOD5     LOAD
     8 ING  VELOCITY OF  BOD5   BOD    PRODUCTION  RATE',5X,'MLVSS')
202  FORMAT(//)
201  FORMAT(I3,2X, I2, I3, 2X,F5.3,5X,F6.3,4X,F5.2,5X,F5.0,5X,F5.0,5X,
    1F5.2,5X,F5.2,6X,F4.1,5X,F5.0,4X,F6.3,5X,F5.3,5X,F5.0)
224    FORMAT(I3,2X,I2,1X,I2,                   2X,F5.0,5X,F5.0,
    15X,F5.0,5X,  F5.1, 5X,F5.1,5X,F5.1,4X,F7.3,4X,F4.1,8X,F3.0)
225   FORMAT('  SUMMARY OF MIXED LIQUOR CONDITIONS'//
    2'  MO  DAY  YR      PH',8X,'T',8X,'TSS',       ,7X, 'VSS',6X,'UPTAKE'/
    322X,     'CENT.',6X, 'MG/L            MG/L    G/G*DAY'//)
226   FORMAT(I3,2X,I2,1X,I2,3X,F4.1,6X,F3.0,6X,F6.0,4X,F6.0,4X,F4.2)
229   FORMAT(I3,2X,I2,1X,I2,                     2X,F5.0,5X,F5.0,
     15X,F5.0,5X,  F5.1,5X,F5.1,5X,F5.1,4X,F7.3,4X,F4.1)
```

TABLE 17(continued)

```
230  FORMAT(1H1, 'TREATABILITY STUDY FOR WASTEWATER ',I3/
    1'  SUMMARY OF INFLUENT CONDITIONS'/
    2'  ALL DATA EXPRESSED AS MG/L EXCEPT AS NOTED'/)
223  FORMAT (1H1, 'TREATABILITY STUDY FOR WASTEWATER',I3/
    1'  SUMMARY OF FILTERED EFFLUENT CONDITIONS'/
    2'  ALL DATA EXPRESSED AS MG/L'/)
231  FORMAT(42X, 'TKN      NO2  +  NO3  TOTAL PHOS', 22X,'DILUTION'/
    1'  MO   DAY  YR', 3X,                    ' BOD5',6X,'COD',6X,'TOC',
    28X,'N',9X,'N',9X,'P',7X,'PHENOL             MBA      WATER/WASTE'//)
232  FORMAT(42X, 'TKN     NO2  +  NO3 TOTAL PHOS', 22X,'            '/
    1' MO   DAY  YR',3X                        'BOD5',6X,'COD',6X,'TOC',
    28X,'N', 9X,'N',9X,'P',7X,'PHENOL       MBA                 '//)
240  FORMAT(1H1,'TREATABILITY STUDY FOR WASTEWATER',I3)
271  FORMAT(1H1,'TREATABILITY STUDY FOR WASTEWATER', I3/
    1' SUMMARY OF RESULTS BASED ON COD'/
    2' LOADING AND REMOVAL VELOCITY ARE EXPRESSED AS LABS COD / LBS MLV
    3SS  * DAY'//
    451X,'FILTERED', 23X,'PERCENT SETTLED'/
    511X,  'VOLUME',13X,'DETENTION  INFLUENT EFFLUENT',
    613X, 'REMOVAL   REMOVAL     EFFLUENT   SLUDGE   GROWTH'/
    7' MO  DAY  YR  OF  UNIT   FLOW  RATE     TIME       COD          COD        LOAD
    8ING  VELOCITY  OF  COD       COD        PRODUCTION RATE',5X,'MLVSS')
281  FORMAT(1H1,'TREATABILTIY STUDY FOR WASTEWATER',I3/
    1'  SUMMARY OF RESULTS BASED ON TOC'/
    2'  LOADING AND REMOVAL VELOCITY ARE EXPRESSED AS LBS TOC  /  LBS MLV
    3SS  *  DAY'//
    451X,'FILTERED',23X,'PERCENT'/
    511X,  'VOLUME',13X,' DETENTION  INFLUENT  EFFLUENT',
    613X, 'REMOVAL    REMOVAL                        SLUDGE    GROWTH'/
    7' MO  DAY  YR  OF  UNIT  FLOW RATE   TIME       TOC      TOC      LOAD
    8ING   VELOCITY   OF   TOC                 PRODUCTION RATE',5X,'MLVSS')
270  FORMAT(I3,2X,I2,I3,2X,F5,3,5X,F6,3,4X,F5.2,5X,F5.0,5X,F5.0,5X,
    1F5.2,5X,F5.2,6X,F4.1,5X,F5.0,4X,F6.3,5X,F5.3,5X,F5.0)
280  FORMAT(I3,2X,I2,I3,2X,F5.3,5X,F6.3,4X,F5.2,5X,F5.0,5X,F5.0,5X,
    1F5.2,5X,F5.2,6 X,F4.1,5X,5X  ,4X,F6.3,5X,F5.3,5X,F5.0)
10      STOP
        END
```

Results of Bench Scale Biological Reactor Studies

Identification of Participants

As agreed at the start of the laboratory investigations, the results for the individual participants are identified only by code. For the treatability studies, the code was the number of the individual industry plus 200: i.e., the code number of industry 40 would be 240. The code used for the integrated wastewater is 510.

Participants Excluded from the Study

No individual treatability studies were conducted on the wastewater from Houdry because the characterization studies had indicated that the BOD_5 concentration was too low for efficient biological treatment.

B. F. Goodrich was also excluded because their plant was not producing a wastewater at the time of the studies.

Computer Output

The summary of results provided by the computer program for each wastewater investigated is not included in this Report, but was submitted as a separate task report.

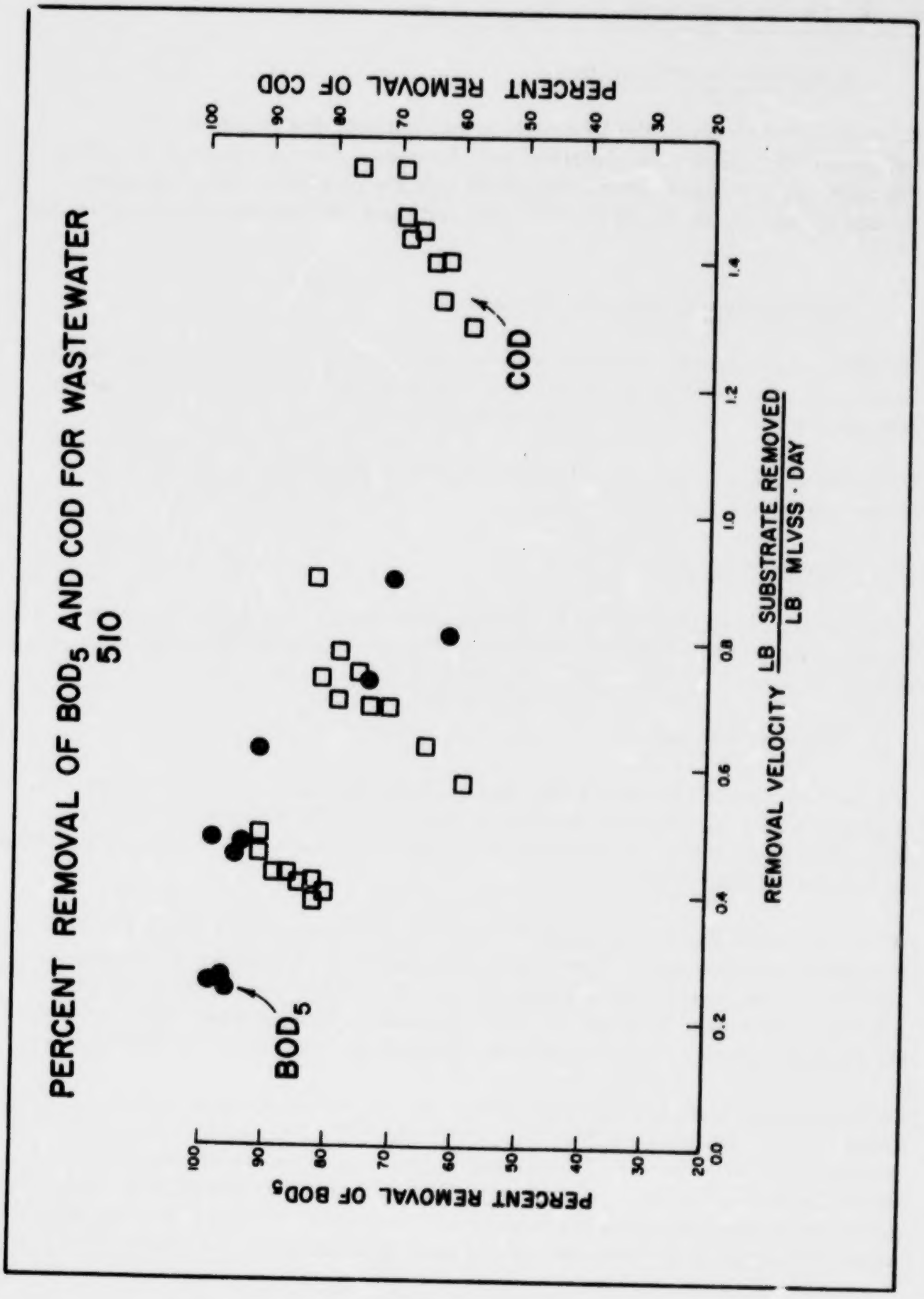
Substrate Removal

The percent removal for both BOD_5 and COD for the integrated wastewater (510) is plotted versus the removal velocity in Figure 10. The same results for the individual participants are presented in Figures 11 through 18. All results are based on filtered effluent samples.

All of the wastewaters investigated resulted in BOD_5 removals in excess of 90 percent at low loadings. (Note: loading is approximately equal to the removal velocity in the lower ranges because of the low effluent concentration of the substrate. At higher loadings the effluent concentration increases and therefore the removal velocity begins to become significantly lower than the loading.)

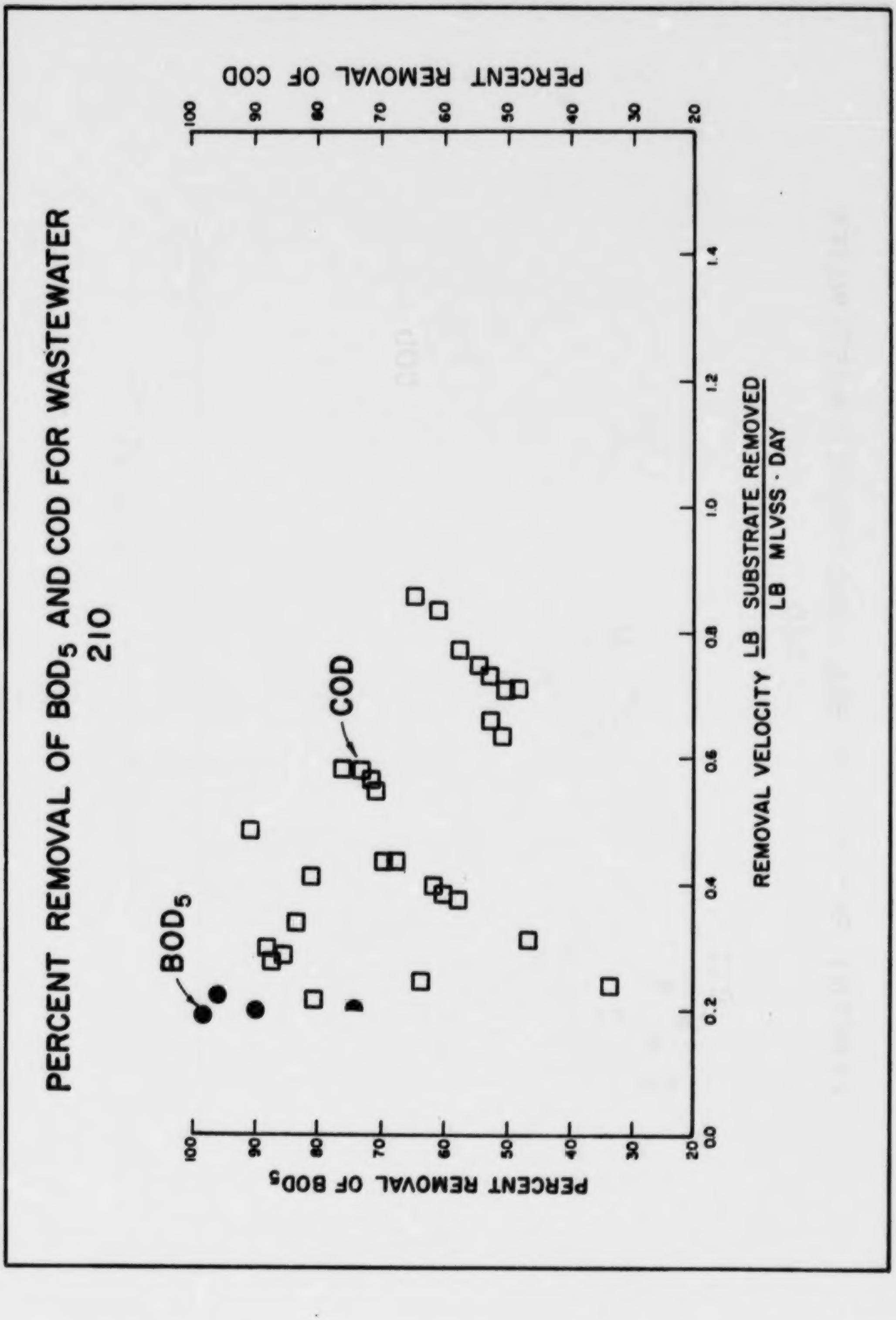
At intermediate and high loadings, results for the individual units varied substantially. Wastewaters 240, 260, 290, and 300 continued to have BOD_5 removals near or in excess of 90 percent at loadings of approximately 0.5 to 0.6 lbs BOD_5/lb MLVSS/day. Wastewaters 220, 230, and 280 experienced fairly uniform decreases in performance as the loading was increased, and the unit treating wastewater 210 could not be operated satisfactorily at loadings above approximately 0.3.

Figure 10



PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER 510

Figure 11



PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER 210

Figure 12

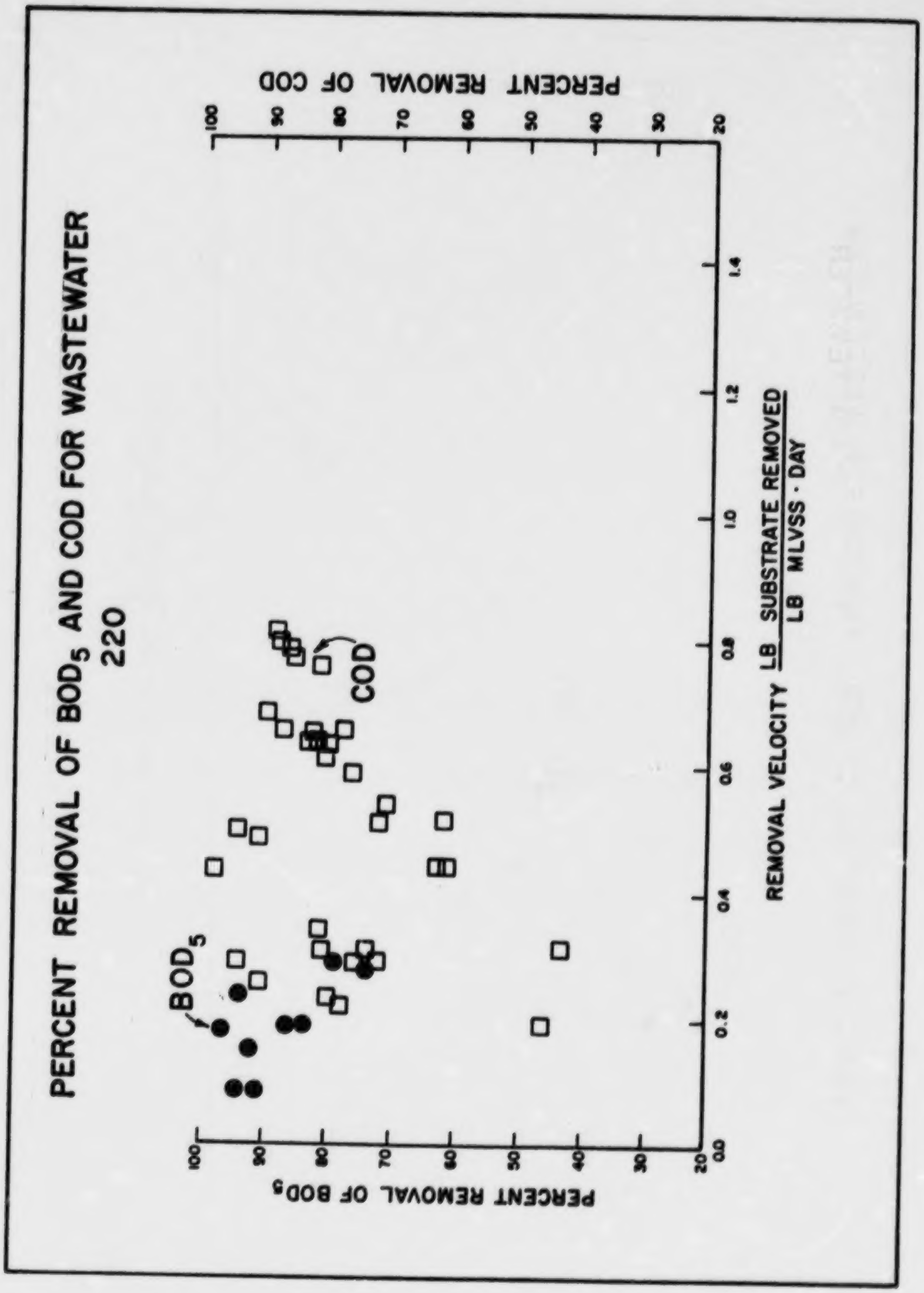
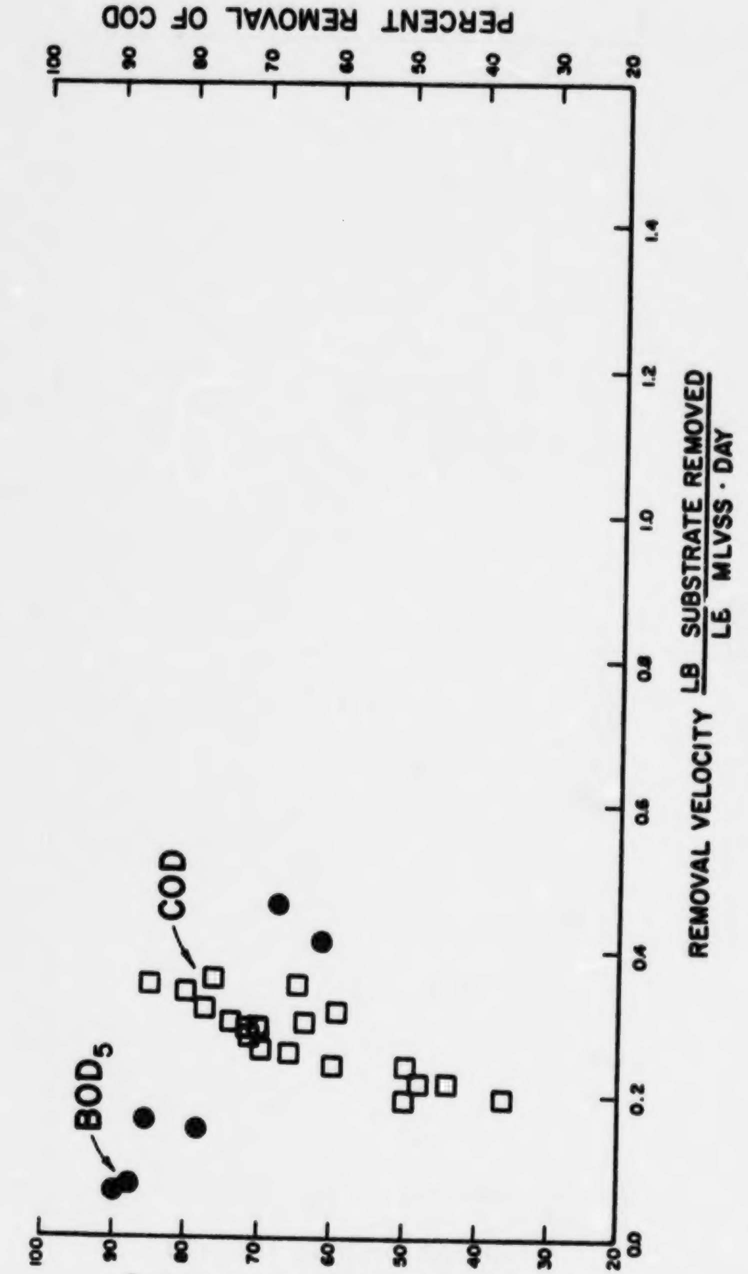
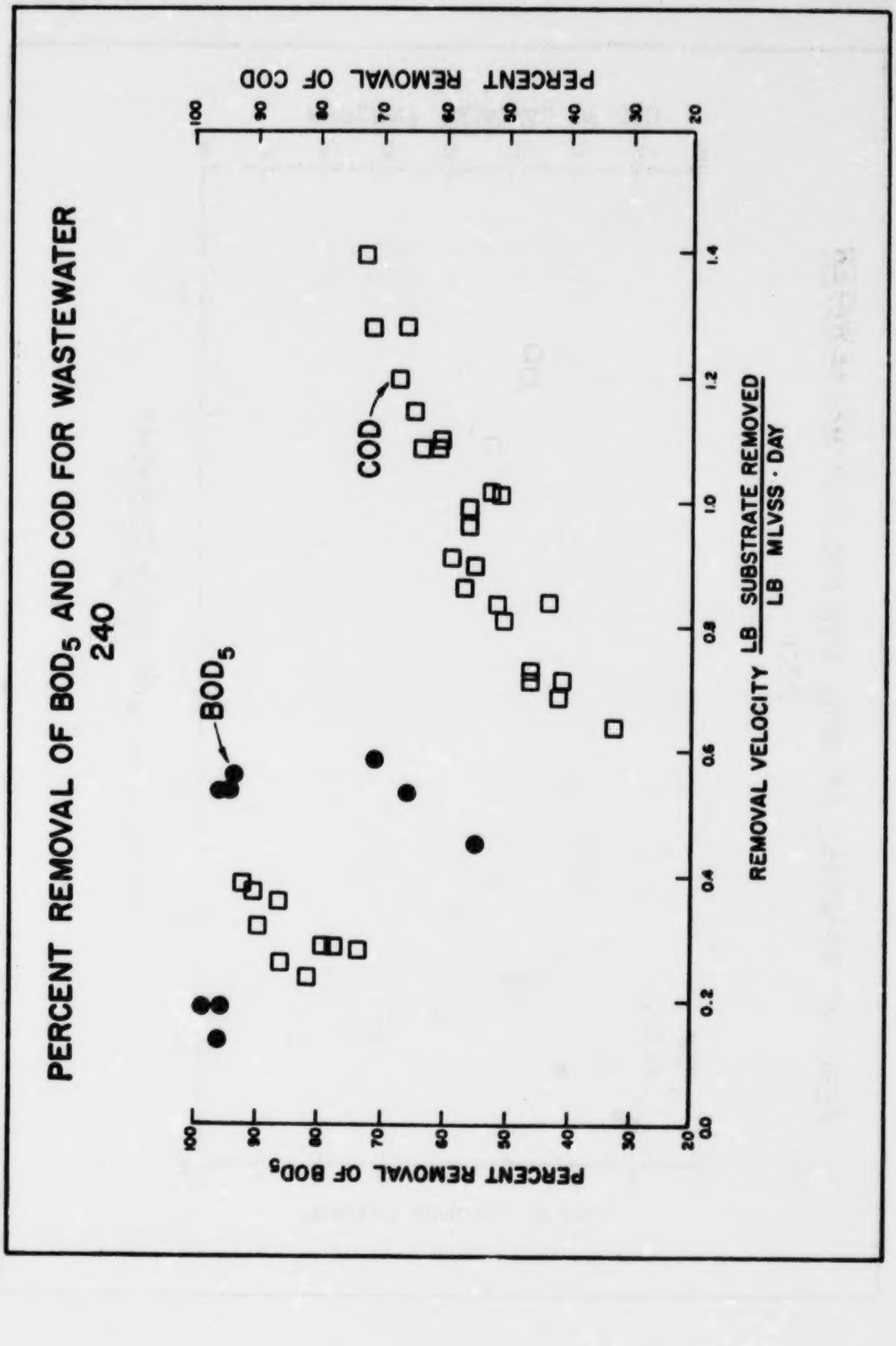

PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER 220

Figure 13

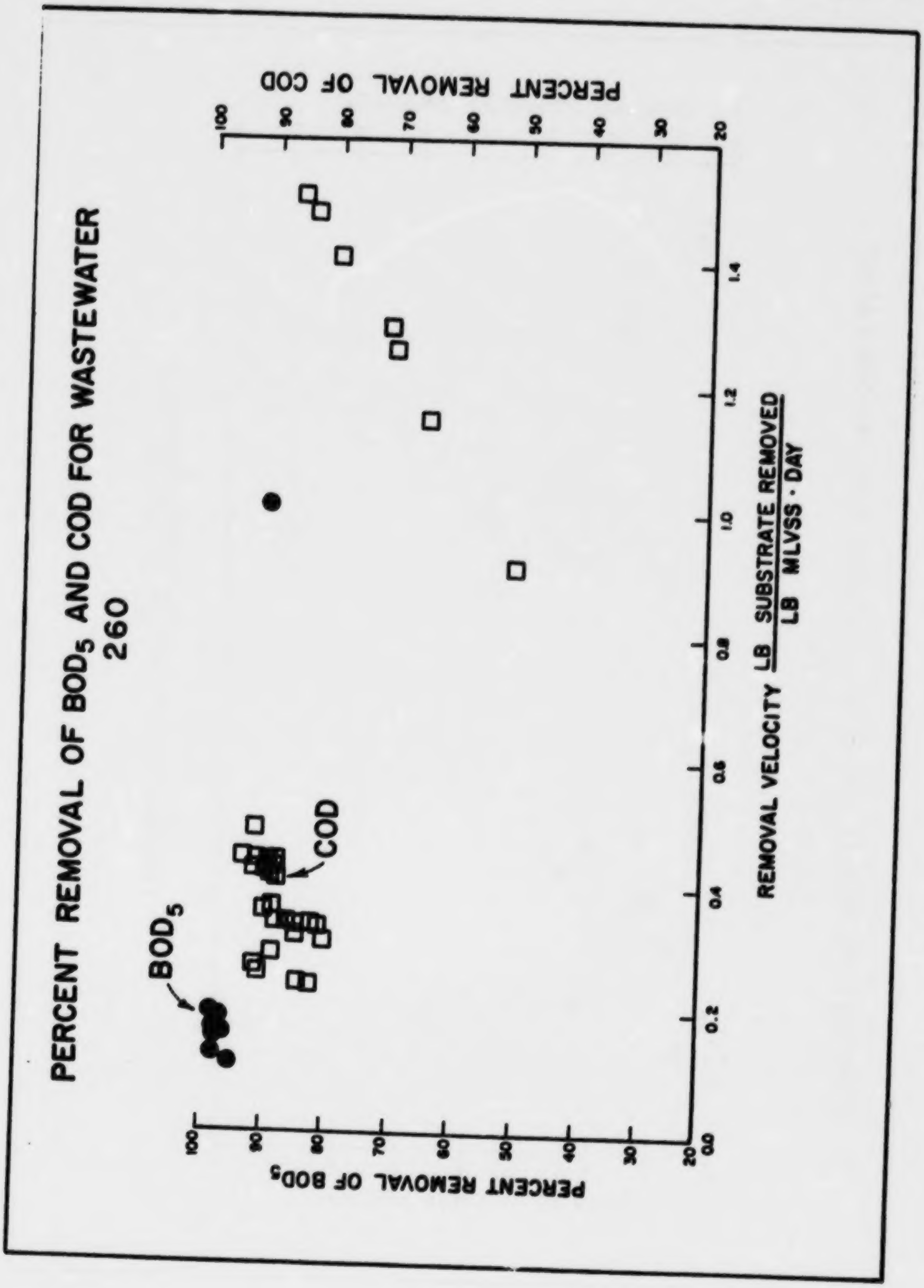
PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER
230





PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER 240

Figure 15



PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER
260

Figure 16

PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER
280

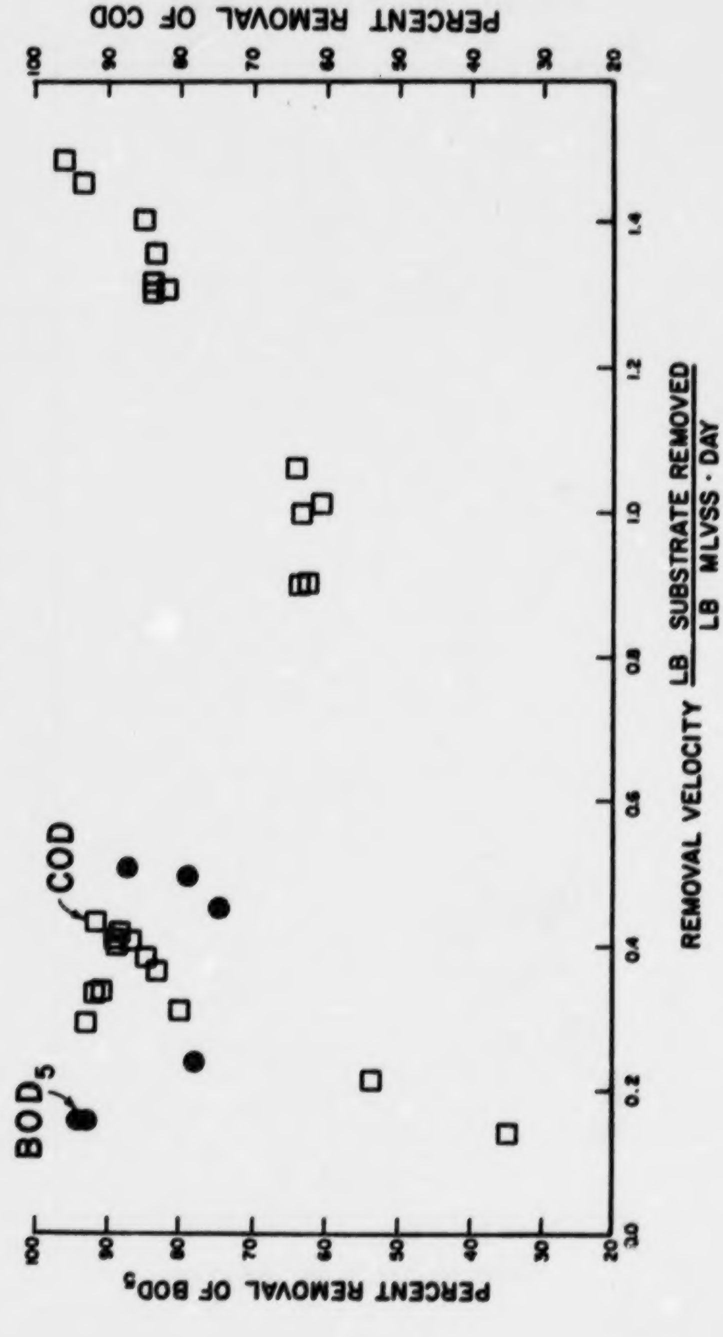
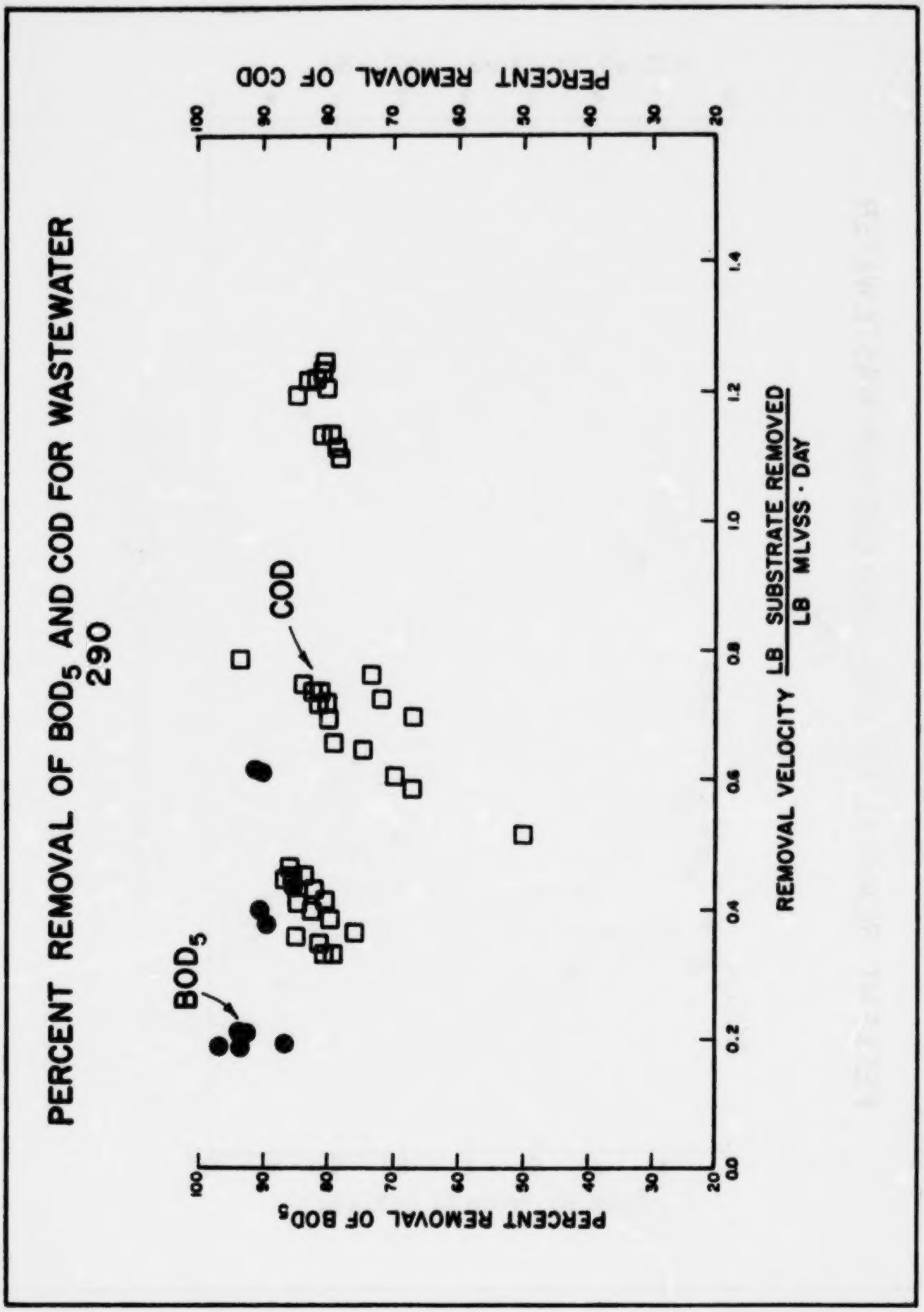


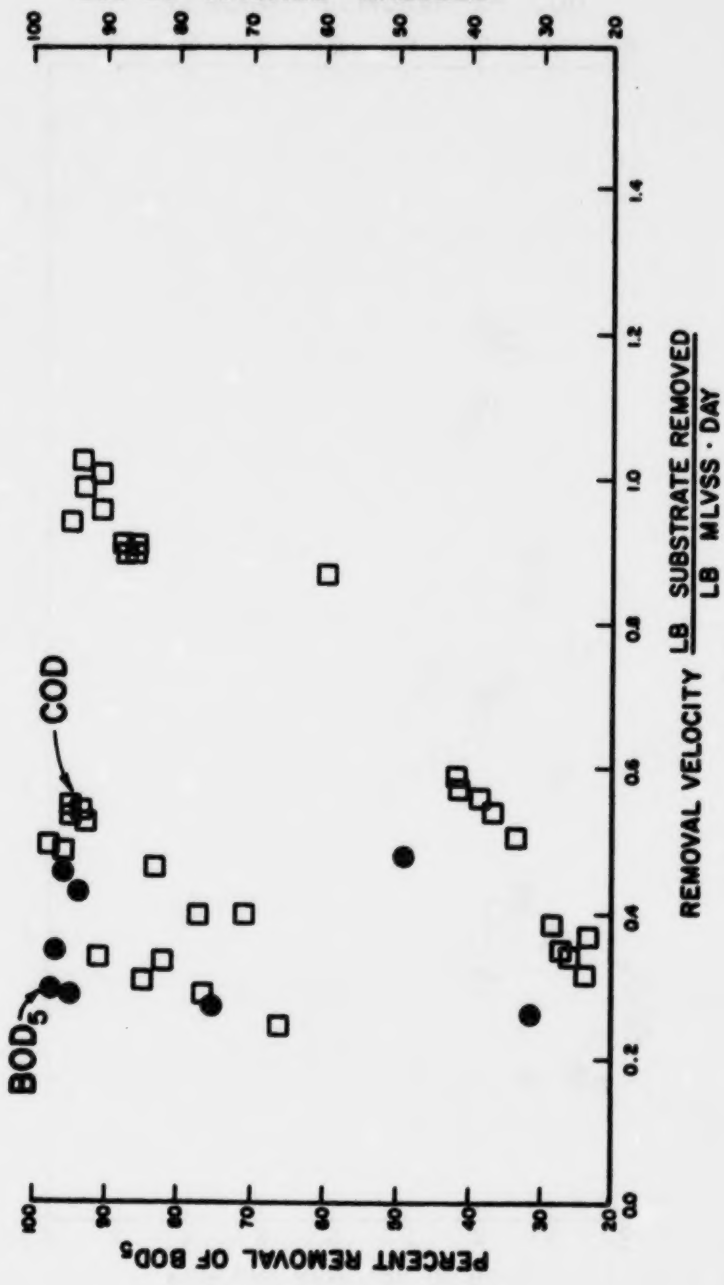
Figure 17



PERCENT REMOVAL OF BOD_5 AND COD FOR WASTEWATER 290

Figure 18

PERCENT REMOVAL OF BOD$_5$ AND COD FOR WASTEWATER 300



The results for the integrated wastewater (510) indicated the removals in excess of 90 percent could be achieved at loadings as high as 0.70. At higher loadings the removals fell off uniformily.

Effluent First State Oxygen Demand (L_a)

The L_a concentration in the filtered effluent for each of the individual wastewaters at the various loadings are summarized in Table 18. For the integrated wastewater (510), the data indicated that the effluent L_a would be 36 mg/l at a loading of 0.50. This would result in 310 pounds of first stage oxygen demand being discharged in the effluent from a regional plant for each million gallons treated.

Substrate Removal Rate

Effluent concentrations for both BOD_5 and COD versus the removal velocity for the integrated wastewater (510) are plotted in Figure 19. The resulting substrate removal rate K, which is the inverse of the slope of the line of best fit, is 0.0316 using BOD_5 as the basis and 0.00725 based on COD. In the latter case, there is an extrapolated, non-degradable COD concentration in the effluent of approximately 30 mg/l, although the actual COD residual value will probably be higher. On the basis of extrapolation, the factor "y" in the previously derived equation for substrate removal (Equation V-5) is 0.2. In the case of BOD_5, there is negligible residual concentration in the effluent and therefore y approximates 0.

To demonstrate the use of the substrate removal equation, assume it was desirable to maintain an effluent BOD_5 concentration of 15 mg/l. Using the above coefficients

$$\frac{S_o - S_e}{X_a t} = KS_e - y = (0.0316)(15) - 0 = 0.475 \frac{\text{lbs } BOD_5 \text{ Removed}}{\text{lb MLVSS day}}$$

Therefore the required removal velocity is 0.475. Assuming also that the influent BOD_5 is 300 mg/l and the MLVSS concentration in the aeration basin will be maintained at 2000 mg/l, the required detention time is as follows:

$$\frac{S_o - S_e}{X_a t} = 0.475$$

$$\frac{300 - 15}{2000\, t} = 0.475$$

$$t = 0.3 \text{ days} = 7.2 \text{ hours}$$

TABLE 18

EFFLUENT FIRST STAGE OXYGEN DEMAND FOR INDIVIDUAL WASTEWATERS AT VARIOUS LOADINGS(a)

Wastewater	(b) Load	(b) Removal Velocity	L_a mg/l	Load	Removal Velocity	L_a mg/l	Load	Removal Velocity	L_a mg/l
510	0.27	0.26	7	0.50	0.47	36	0.70	0.63	71
210	0.26	0.20	125						
220	0.15	0.14	6	0.23	0.19	16	0.37	0.27	31
230	0.08	0.07	9	0.19	0.16	10			
240	0.14	0.13	6	0.60	0.56	47	0.82	0.58	105
260	0.29	0.29	4	0.26	0.26	13			
280	0.57	0.51	42						
290	0.22	0.21	64	0.68	0.62	200			
300	0.31	0.29	30	0.85	0.26	3400			

(a) First stage oxygen demand (L_a) determined in accordance with DRBC publication dated June 1968.

(b) The units for load and removal velocity are lbs BOD_5/lb MLVSS/day.

Therefore the required detention time for the assumed conditions, namely, 95 percent BOD_5 removal, is 7.2 hours. Scale-up factors and temperature factors dictate a longer detention requirement for a full-scale system, however.

It is to be noted that these results are based on filtered effluent samples. Also, as can be seen in Figure 19, Equation V-2 for BOD_5 applies only to removal velocities below 0.60. Above this point, the data were scattered.

Sludge Production and Oxygen Requirements

Oxygen uptake rates and sludge growth rates are plotted versus the BOD_5 removal velocity for each wastewater in Figures 20 through 28. The same data using COD as a basis for the removal velocity are presented in Figures 29 through 37.

These graphs were used in determining the kinetic coefficients in the previously derived mathematical expressions for sludge production and oxygen requirements. The resulting coefficients are summarized in Table 19 for the BOD_5 basis and the COD basis.

Sludge Production

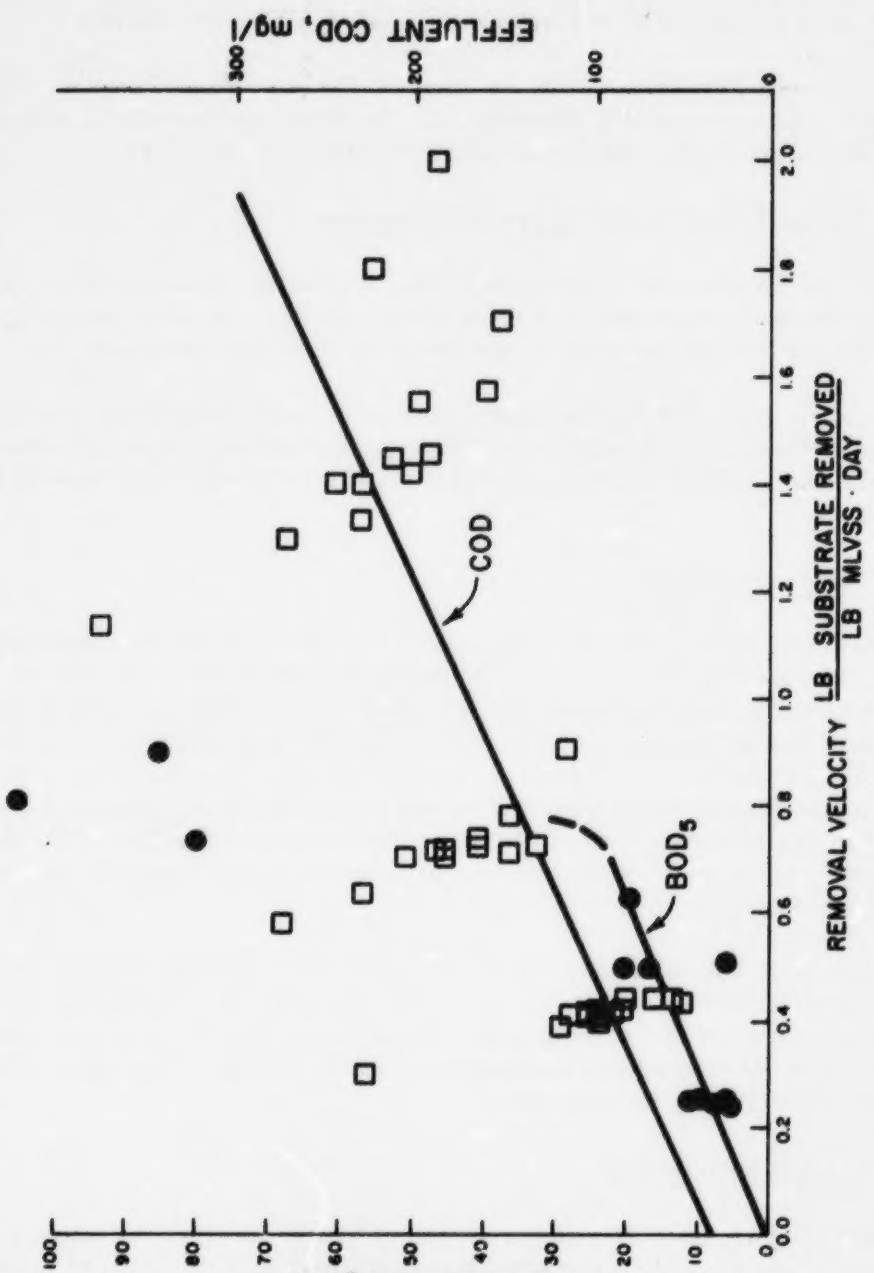
In some cases, particularly for wastewaters 210, 230, and 300 the scatter of points was such that the coefficients could not be determined. The data for the remaining wastewaters indicated that the factor "a", which is the amount of biological sludge produced for each pound of substrate removed, was consistently low. The "a" value of 0.19 derived from the combined wastewater treatability study is significantly lower than that normally experienced for municipal and industrial wastewaters. It should be recognized that the reliability of sludge production values from bench scale studies is low because of the physical limitations of the testing approach.

However, more definitive data was developed from the subsequent pilot plant studies as described in Section VI, indicative that the sludge production rate is in fact lower than that normally reported. Based on the data from the pilot plant studies, it is anticipated that approximately 200 to 300 lbs of biological sludge per day per MGD will be generated.

Oxygen Requirements

The data cited in Table 19 indicate oxygen utilization coefficients which are similar to those normally reported for biological treatment of industrial wastewaters. Applying Equation (V-3), it is estimated that approximately 1,800 lbs of oxygen would be required per MGD treated. The subsequent pilot plant studies described in Section VI closely substantiate this data although somewhat higher values were

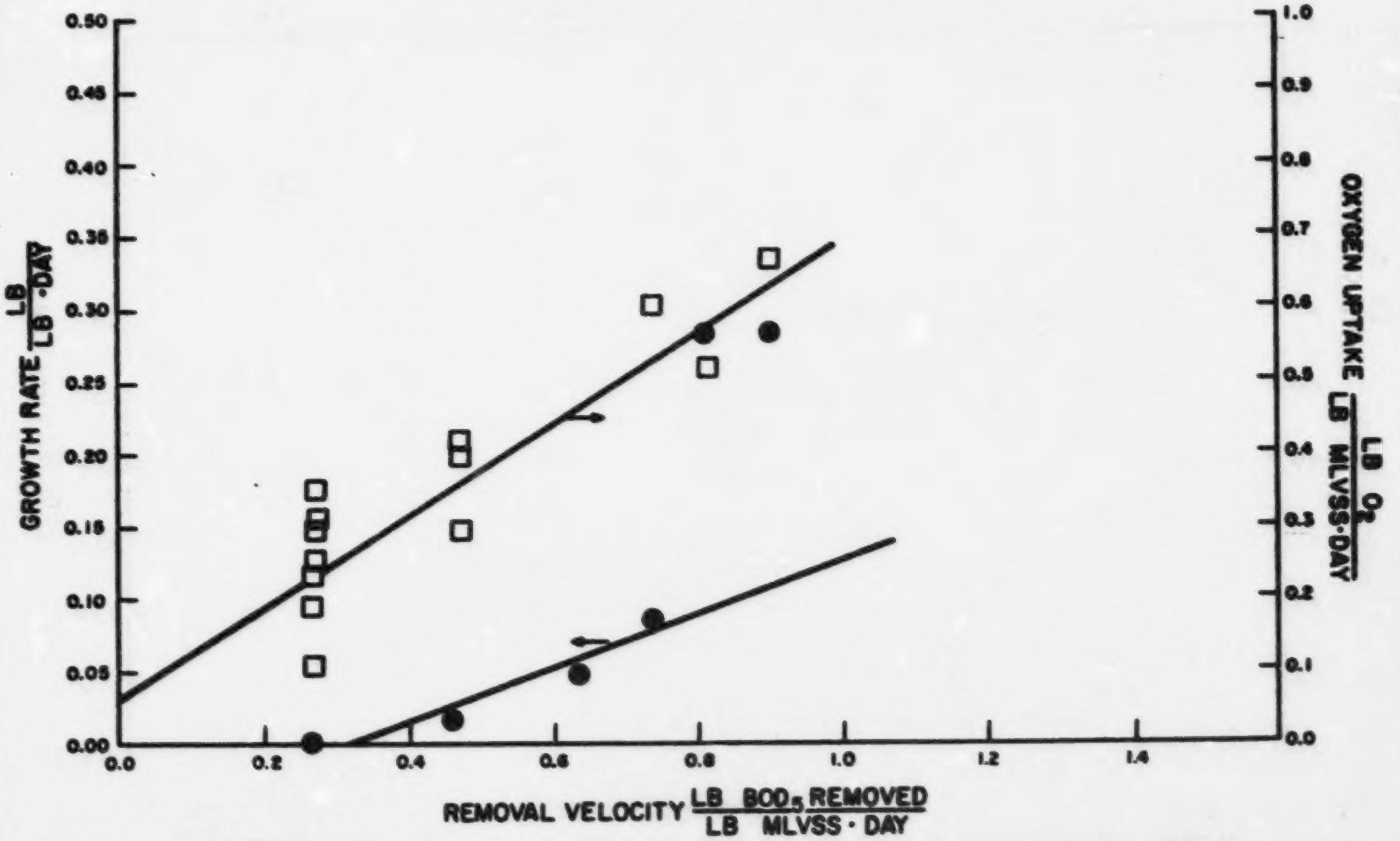
Figure 19



FILTERED EFFLUENT CONCENTRATIONS FOR THE INTEGRATED WASTEWATER

Figure 20

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 510



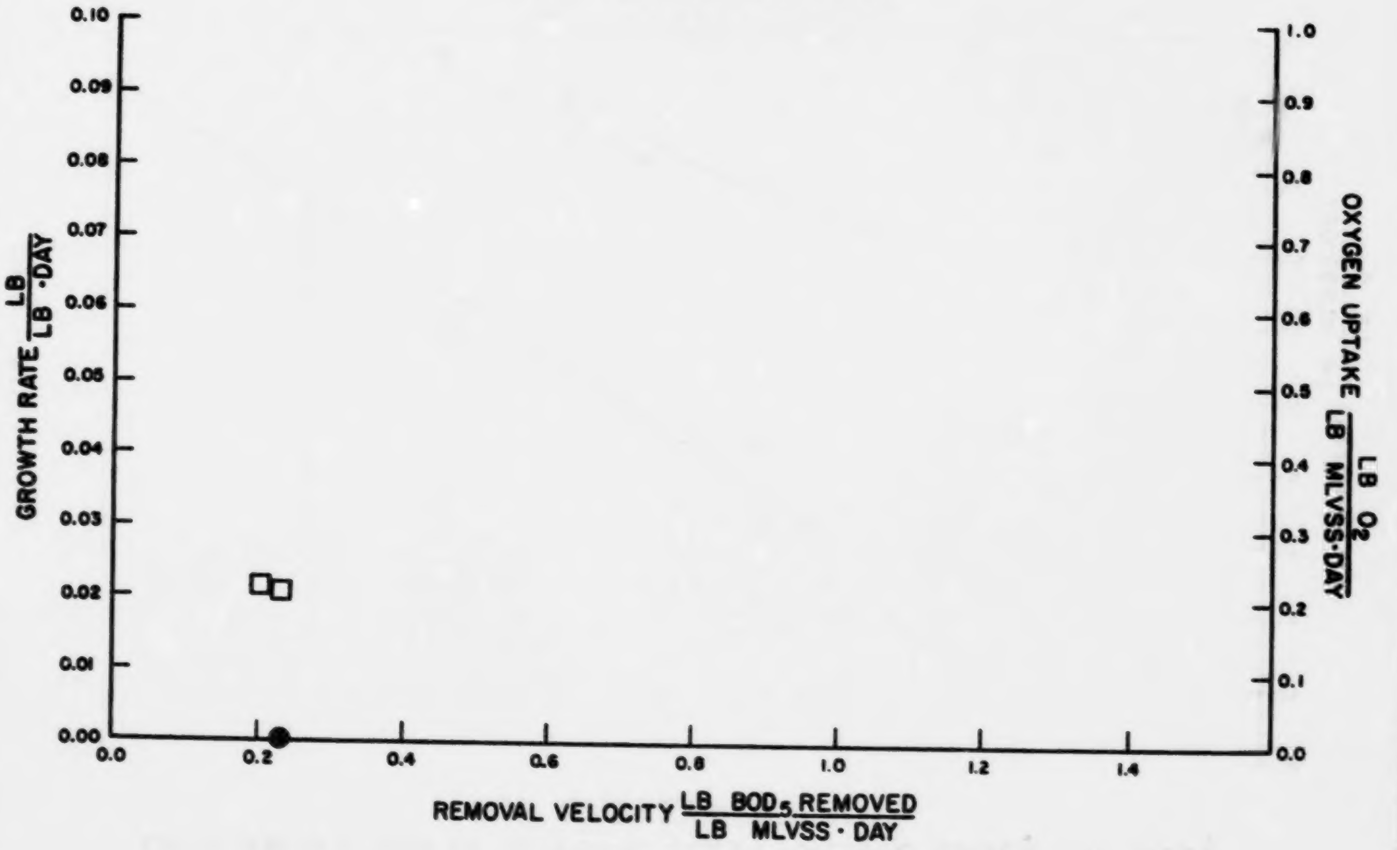


Figure 21

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 210

Figure 22

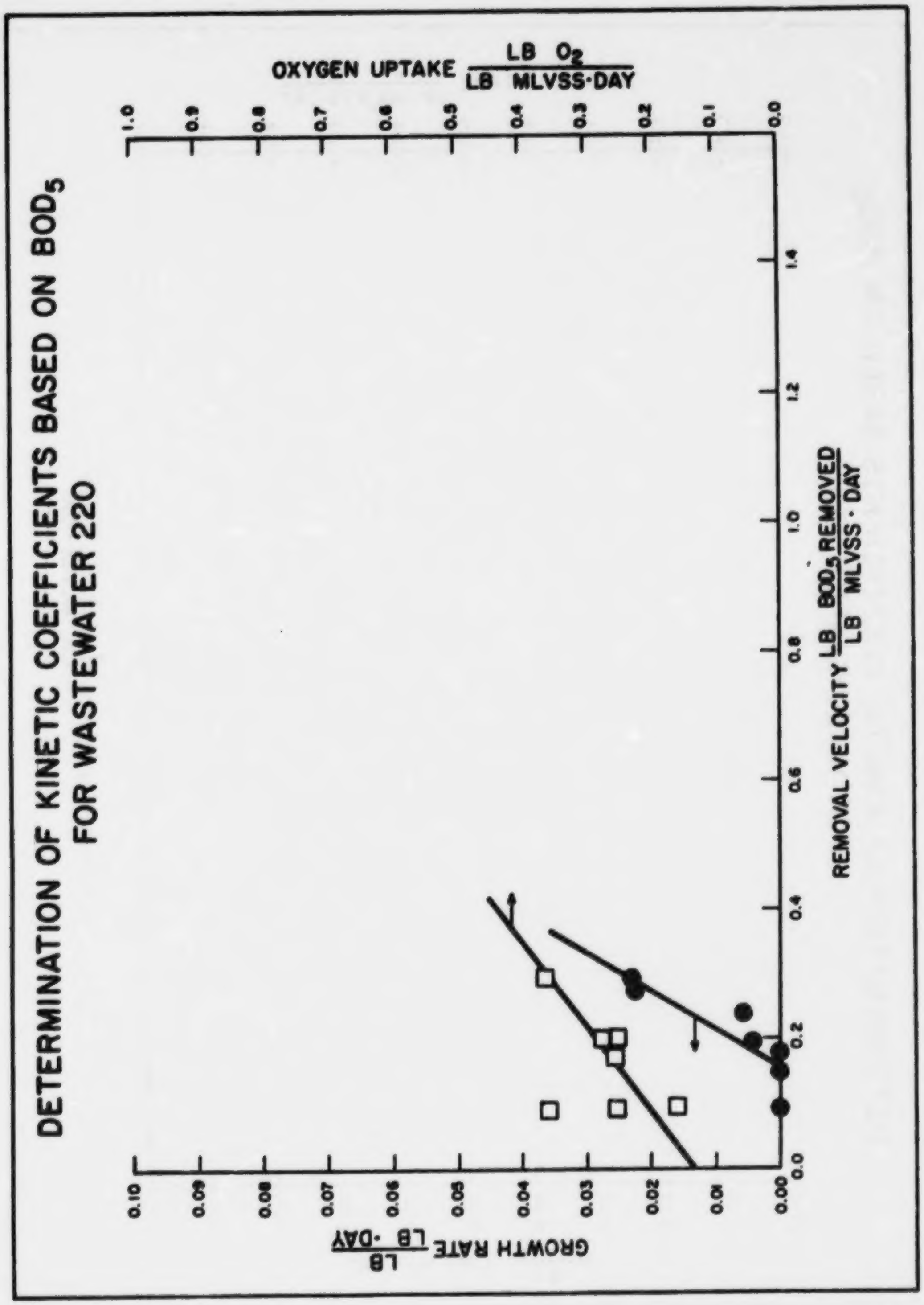
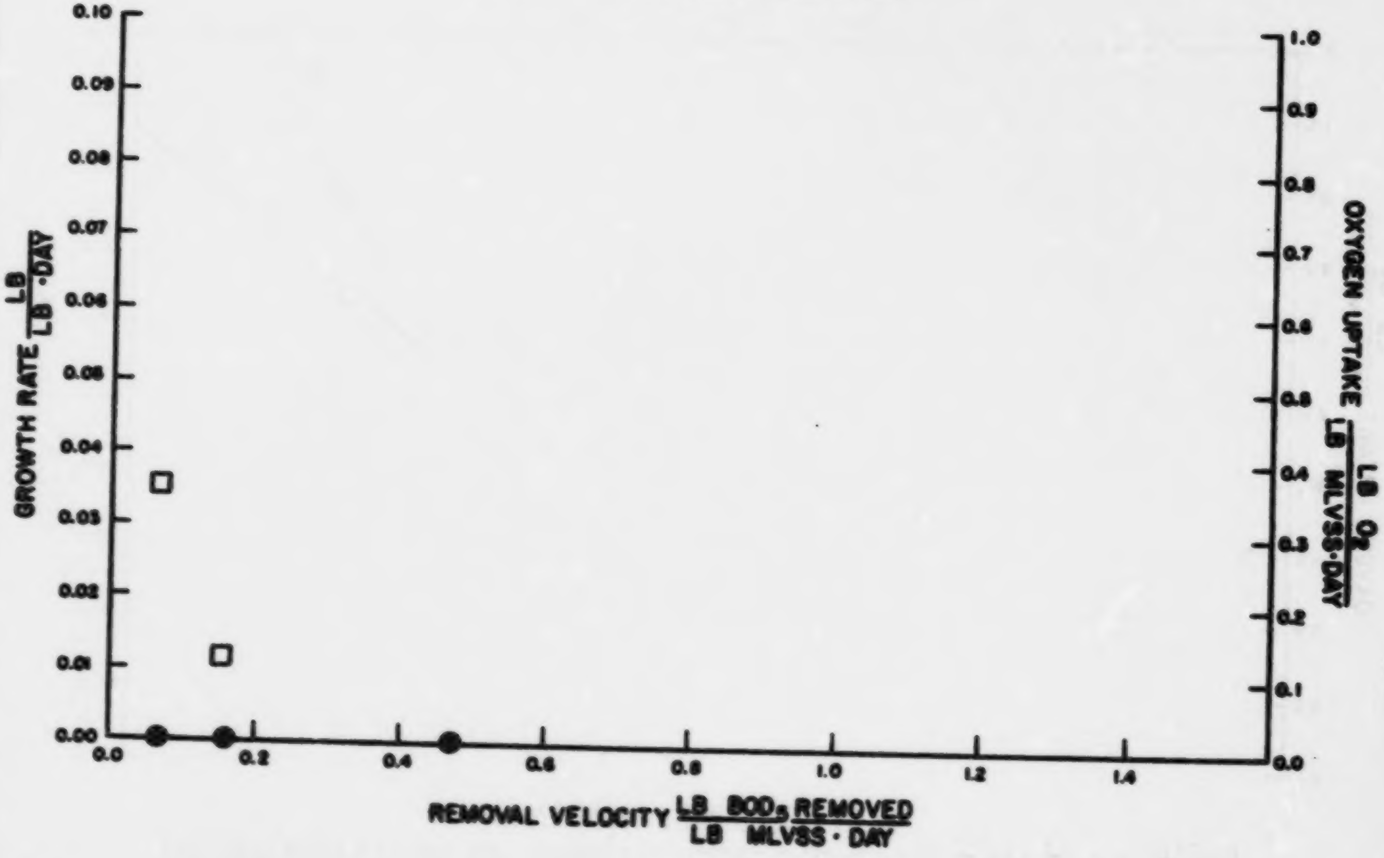

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 220

Figure 23



DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 230

Figure 24

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 240

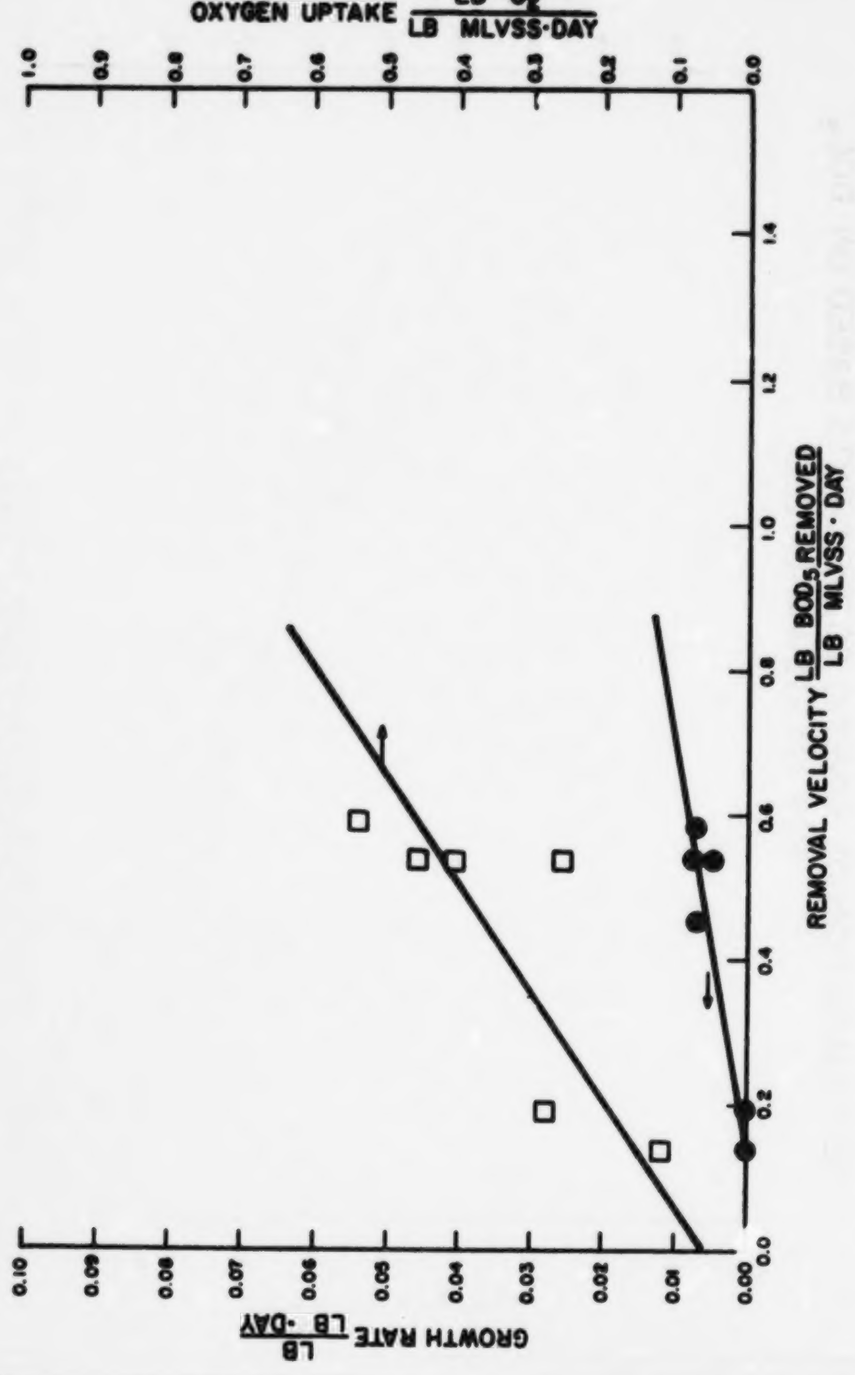


Figure 25

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 260

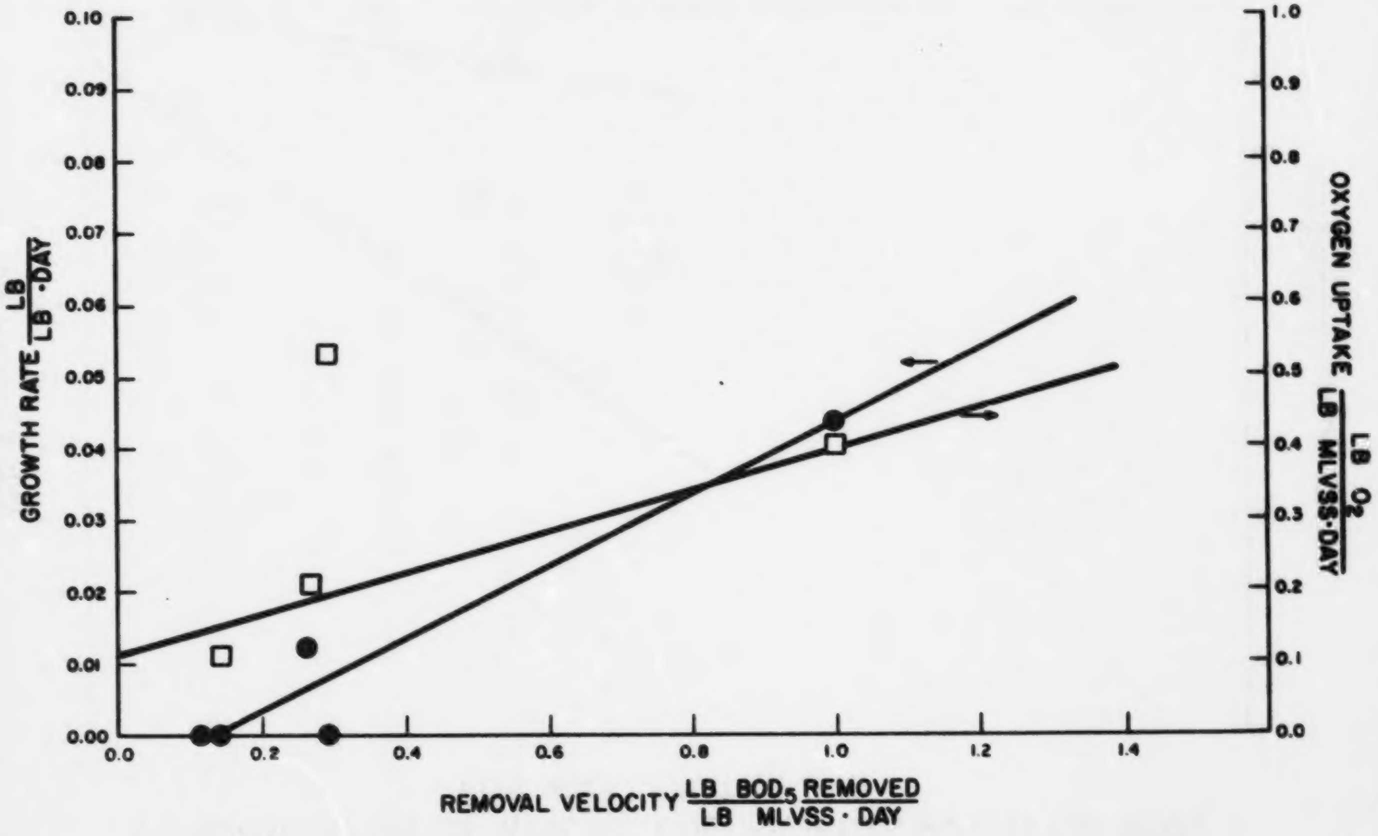


Figure 26

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 280

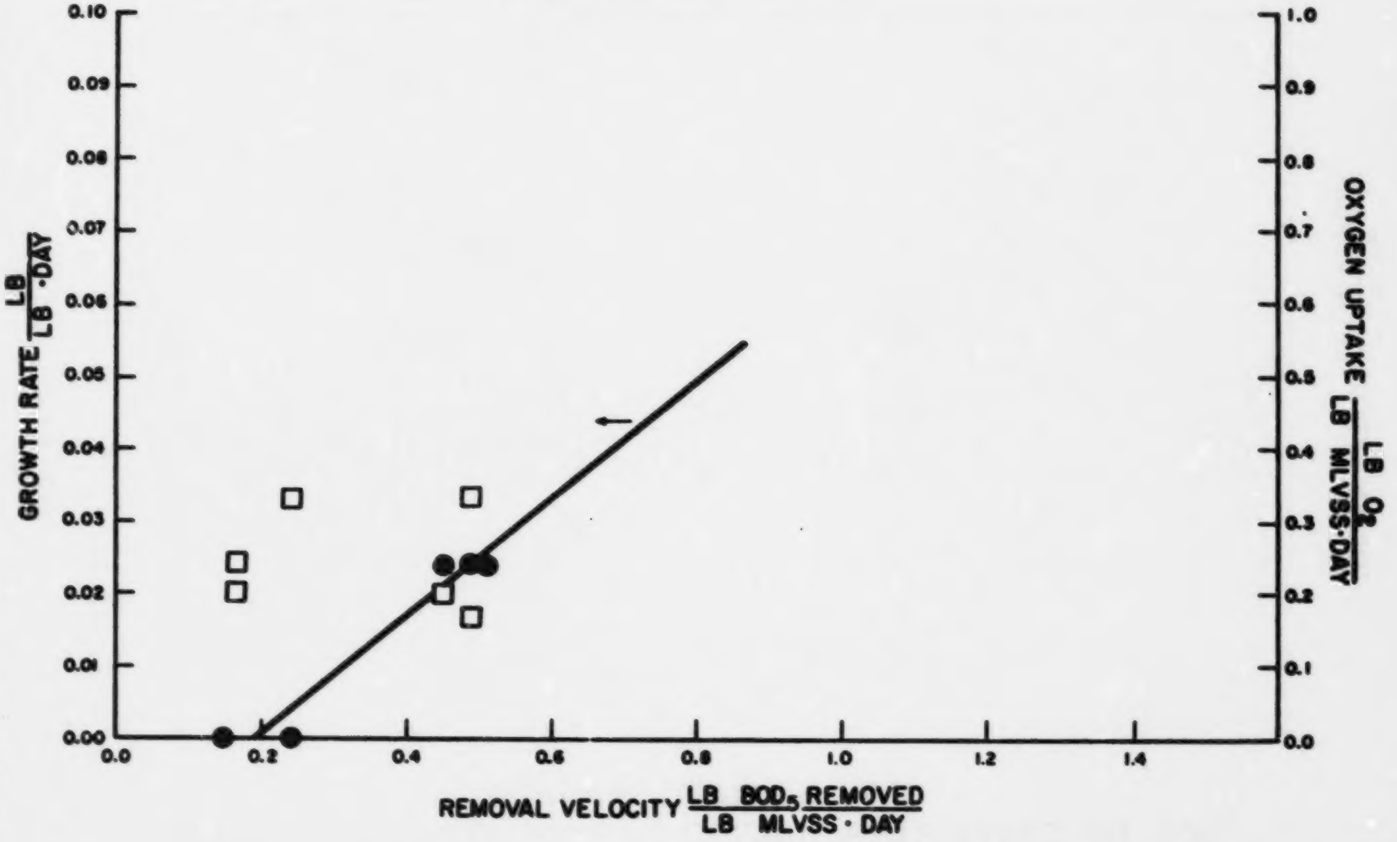


Figure 27

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5 FOR WASTEWATER 290

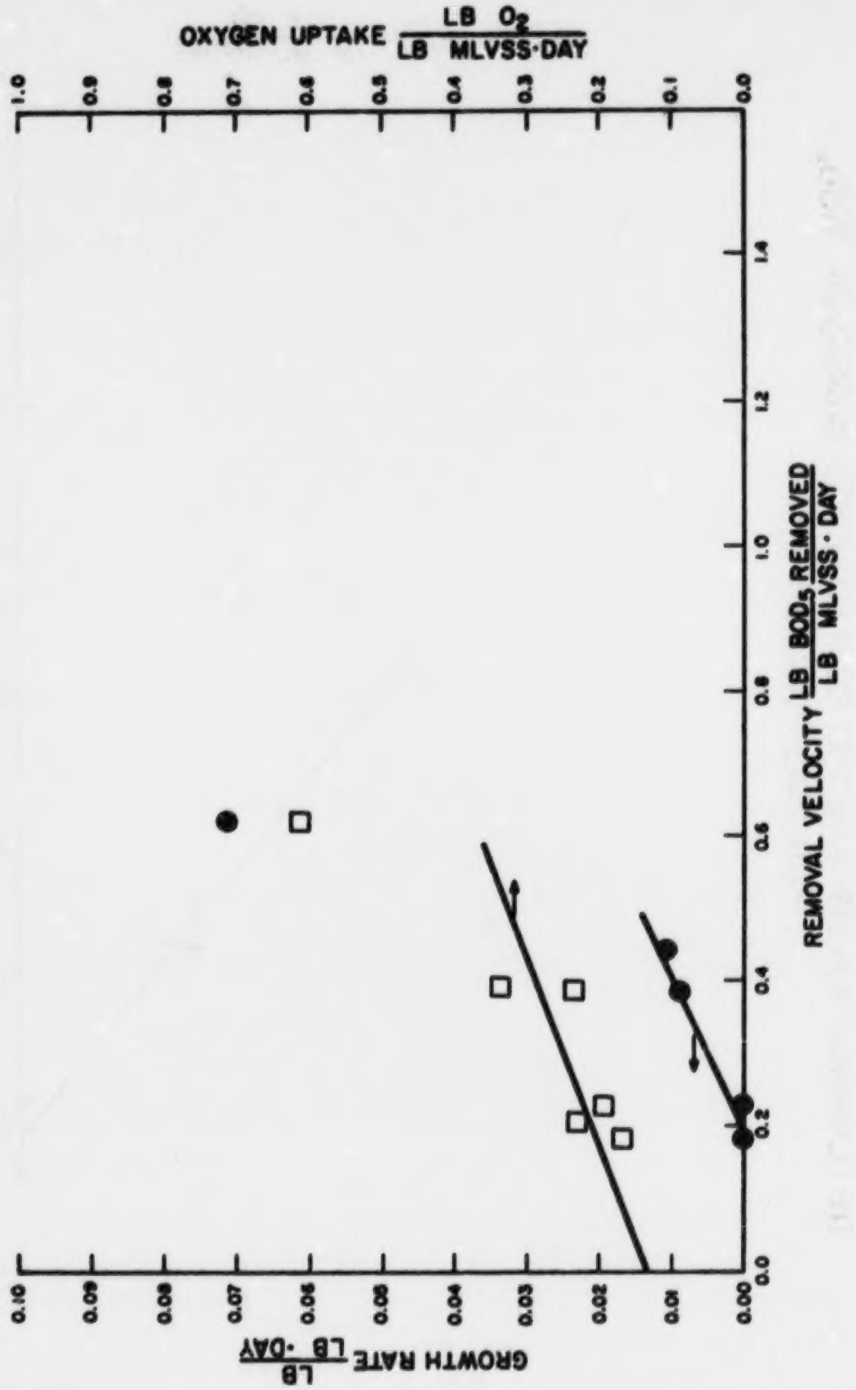


Figure 28

DETERMINATION OF KINETIC COEFFICIENTS BASED ON BOD_5
FOR WASTEWATER 300

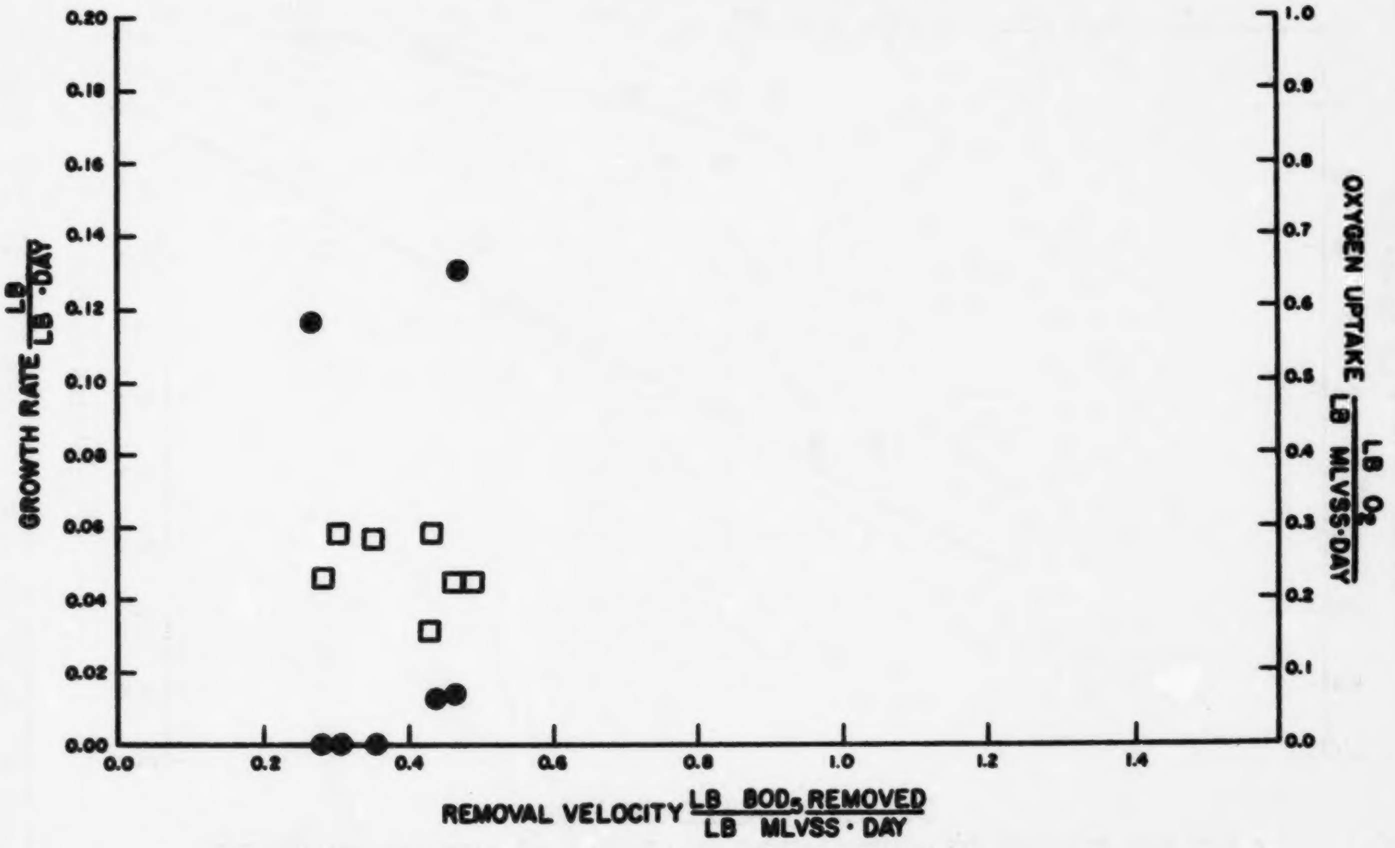


Figure 29

DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD FOR WASTEWATER 510

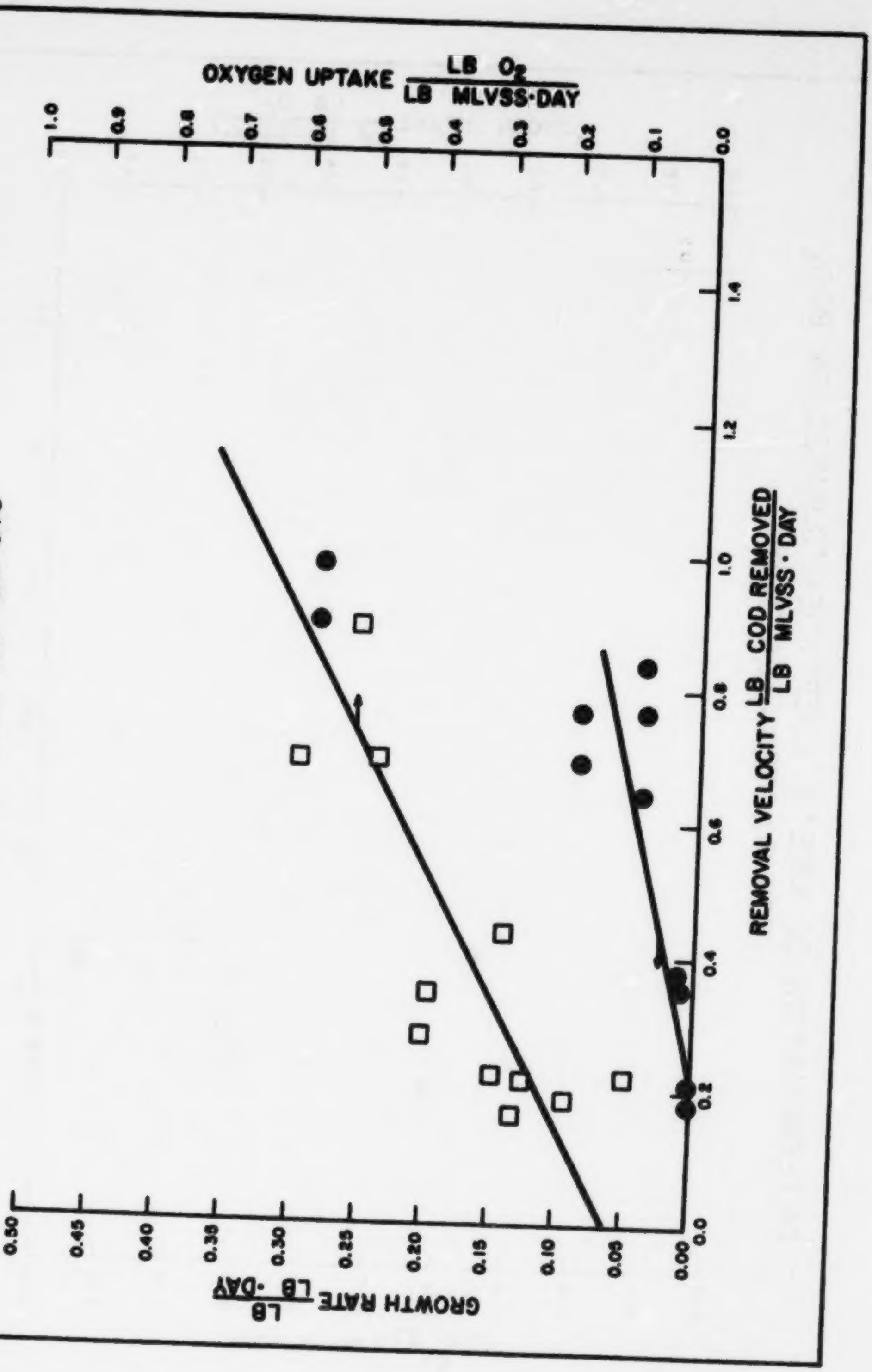


Figure 30

DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD FOR WASTEWATER 210

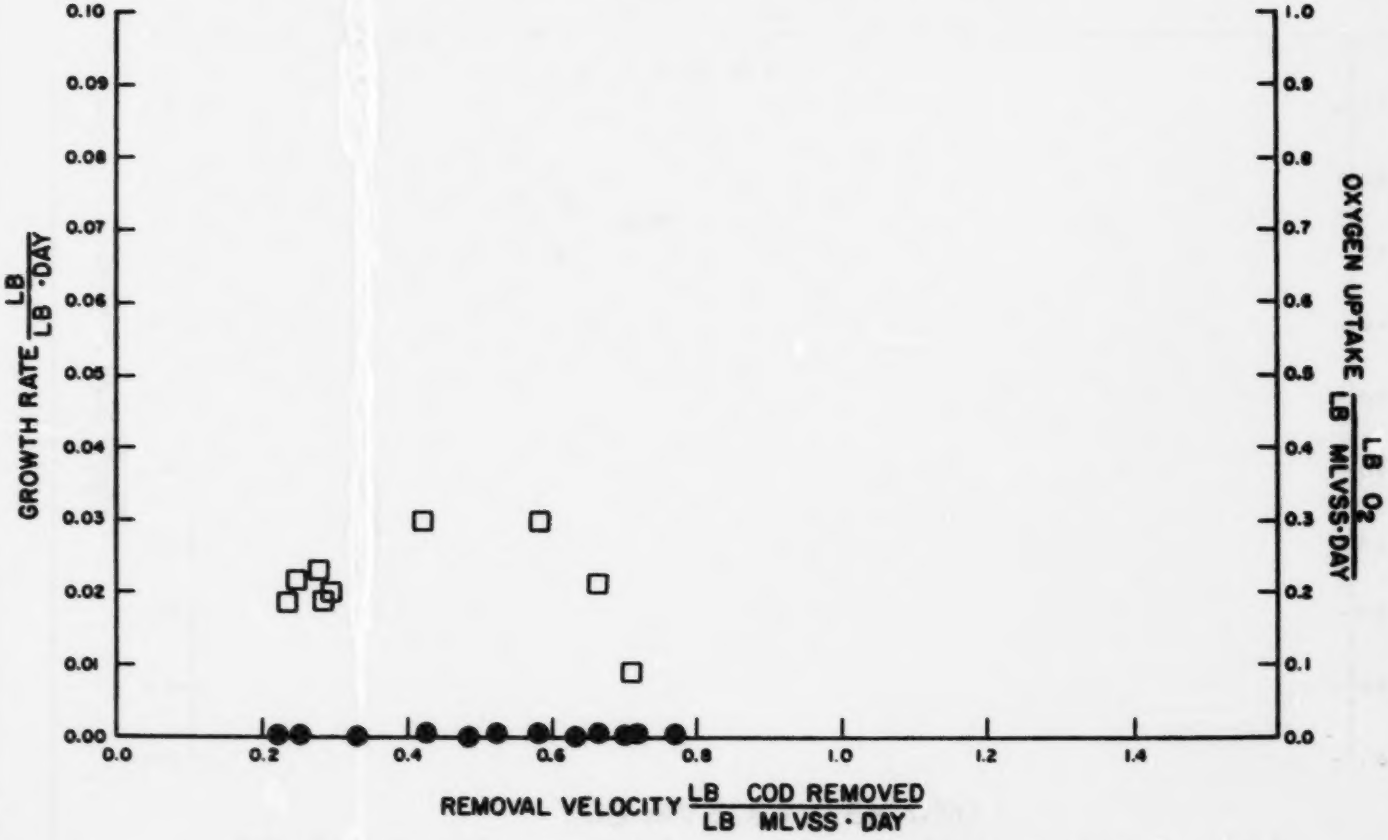


Figure 31

DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD
FOR WASTEWATER 220

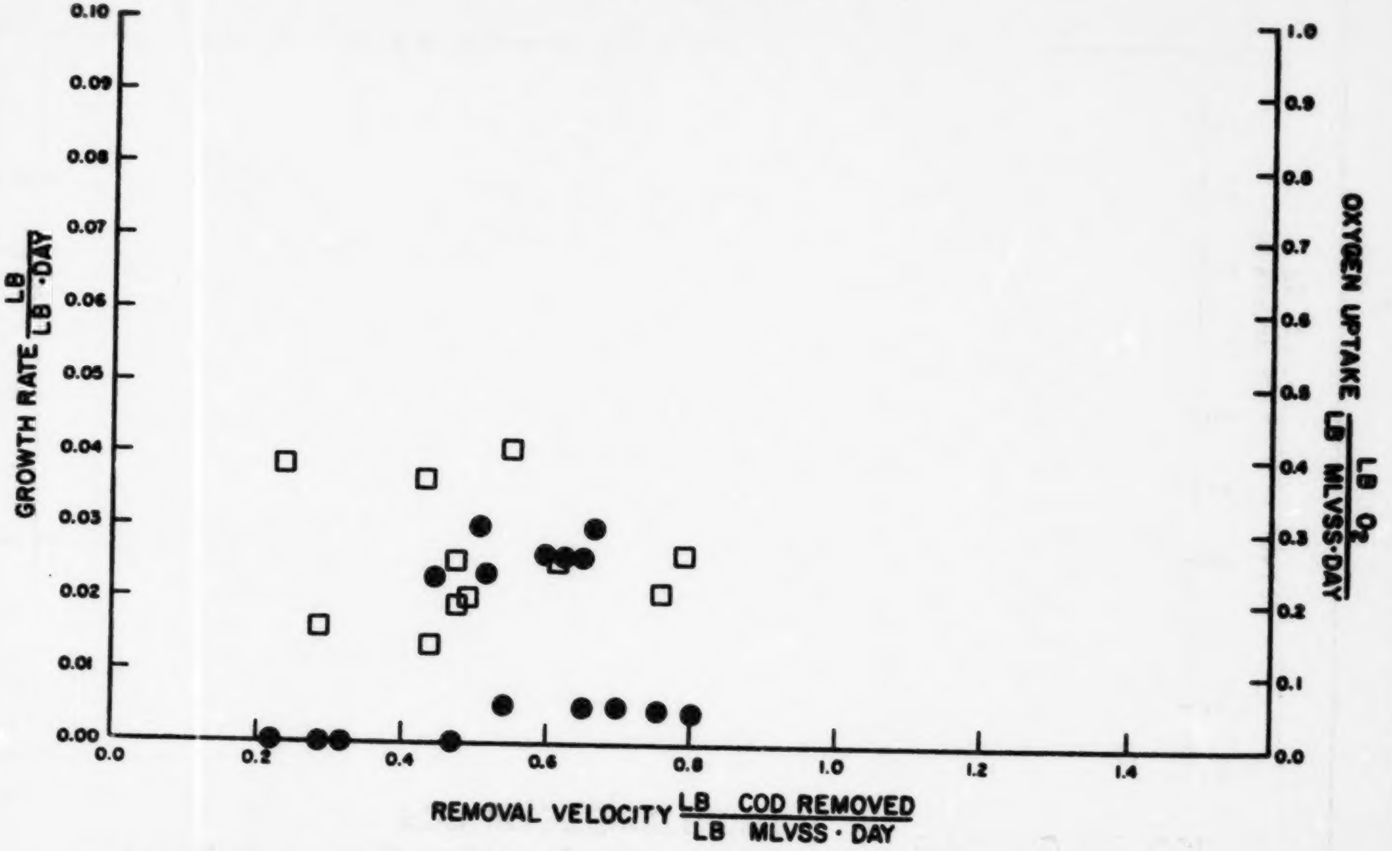


Figure 32

DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD FOR WASTEWATER 230

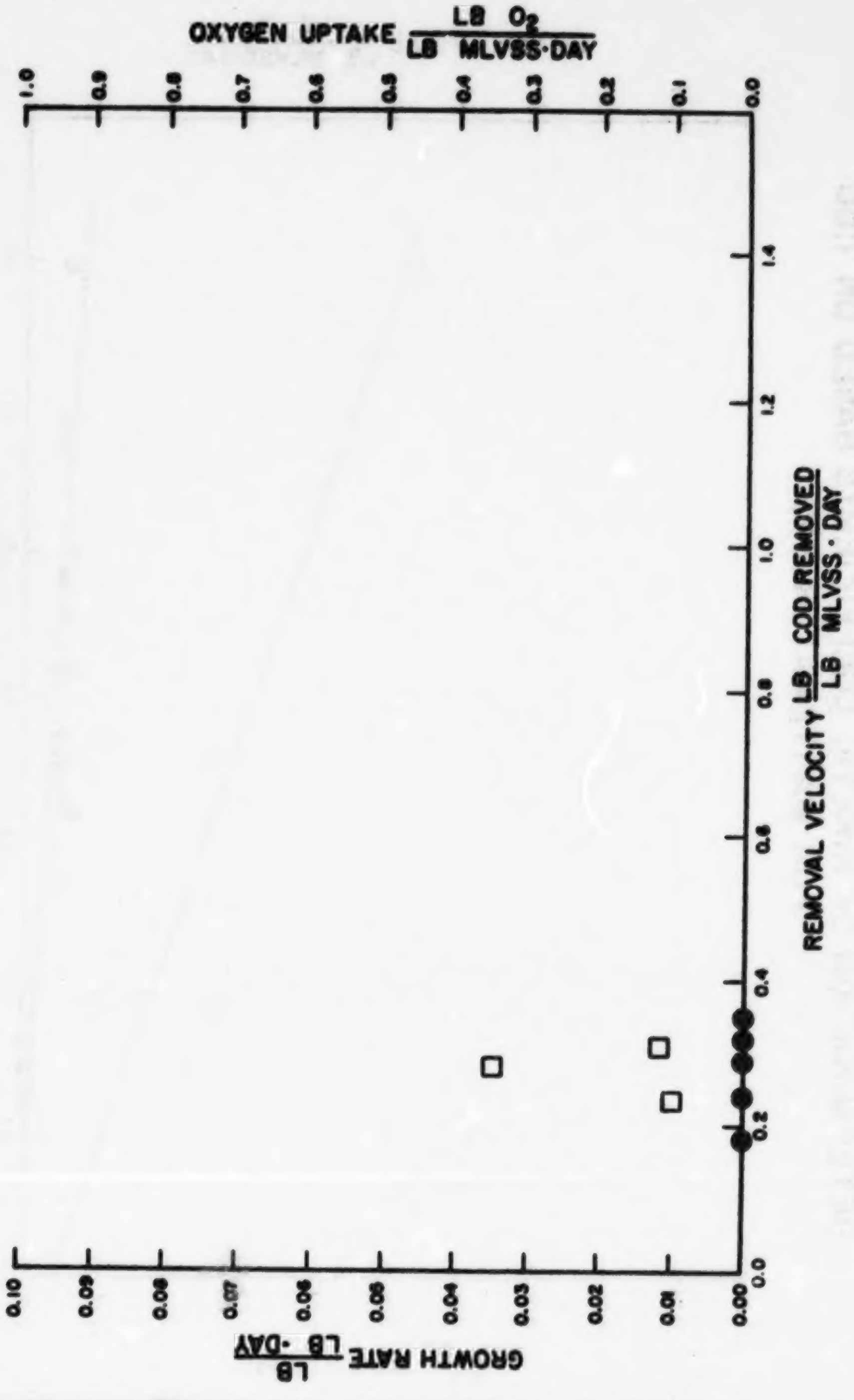


Figure 33

DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD
FOR WASTEWATER 240

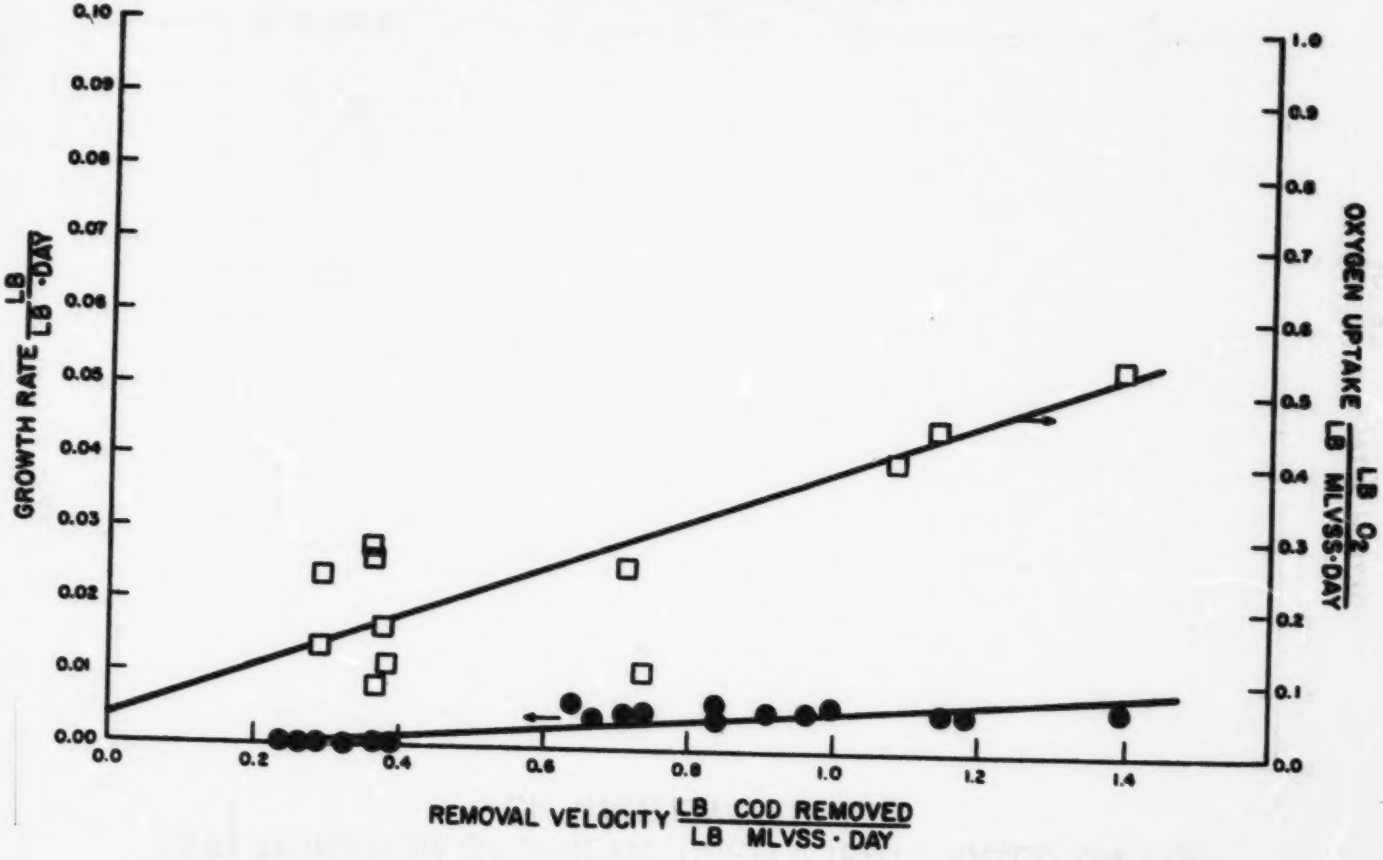
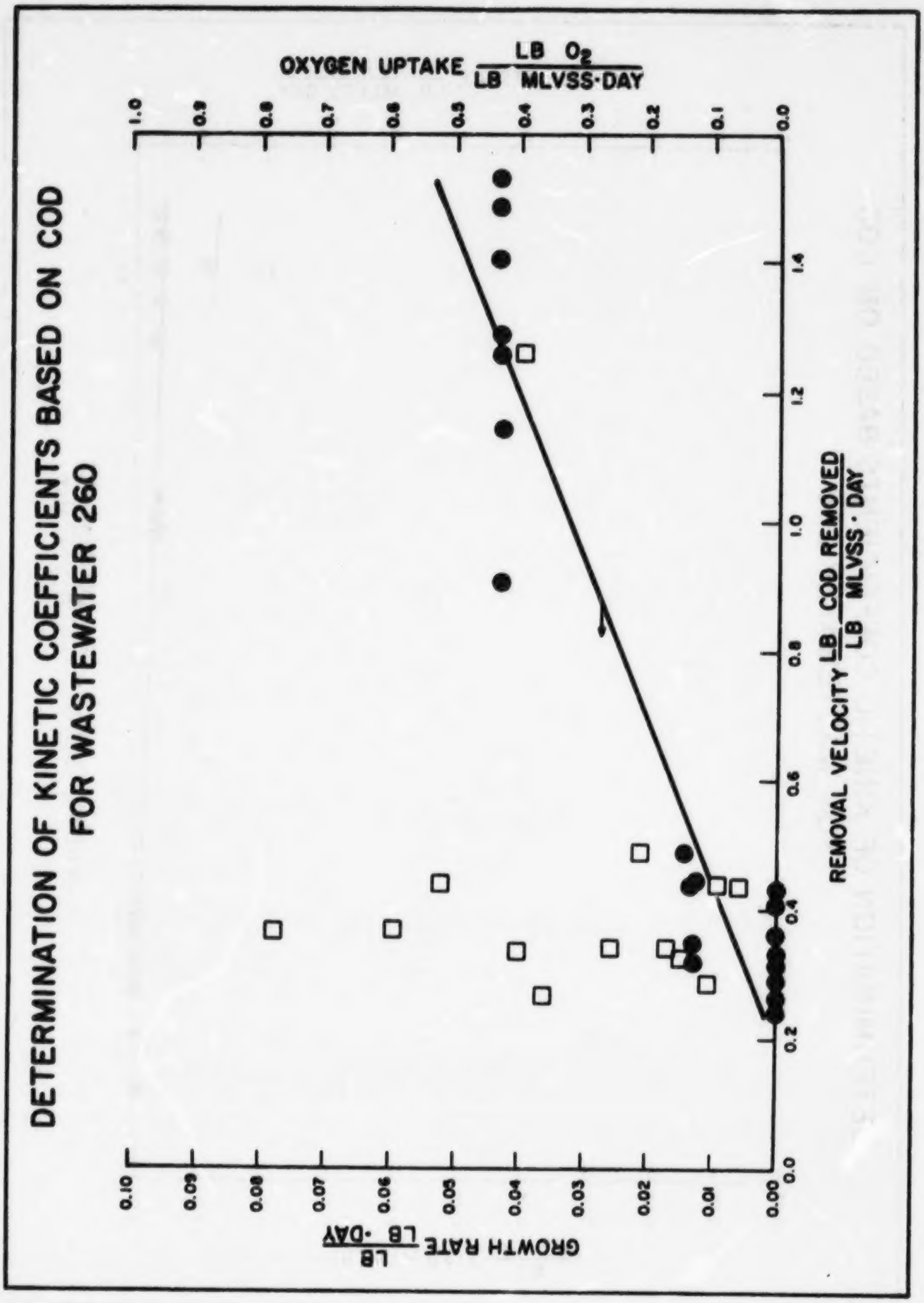
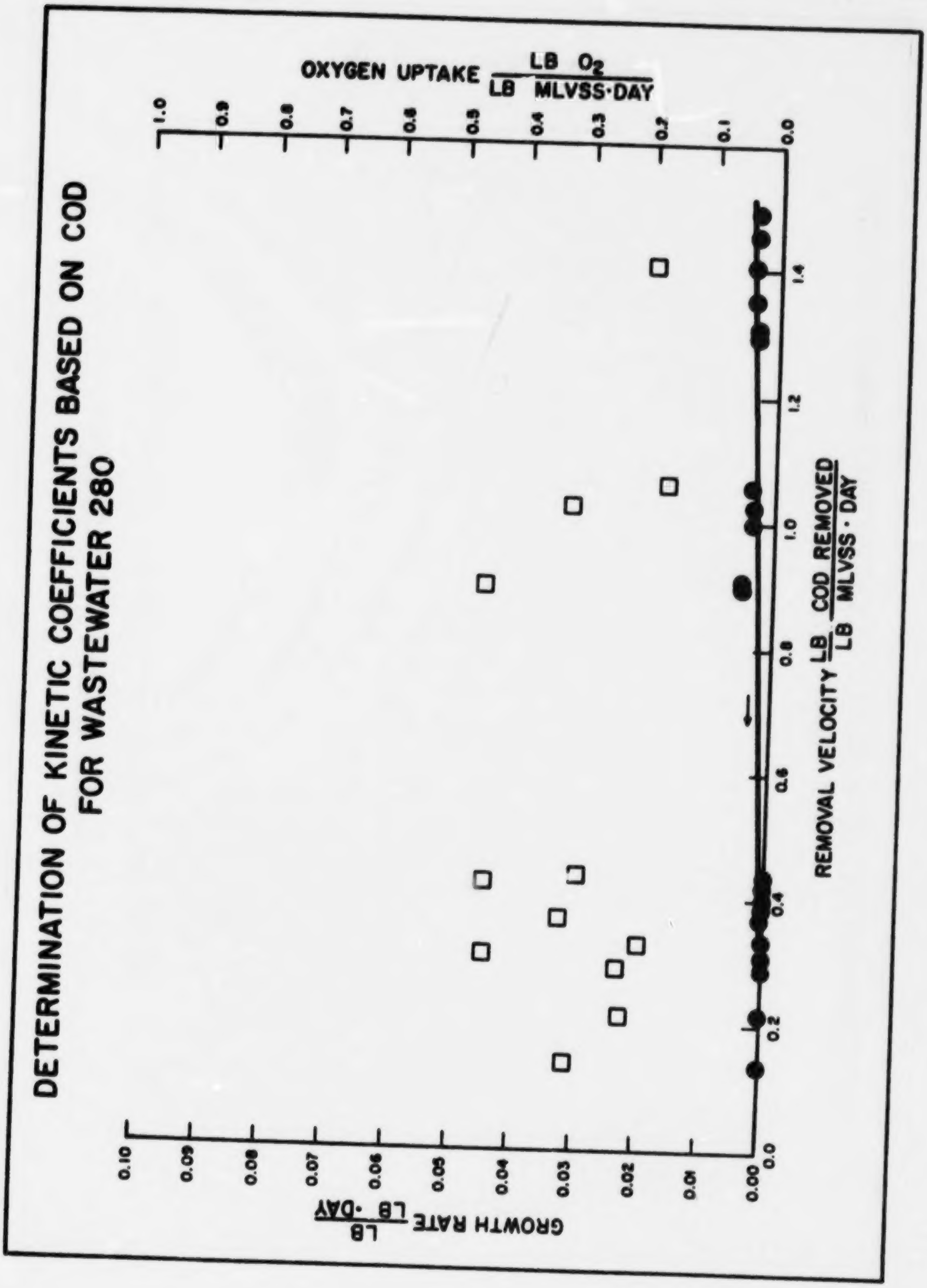


Figure 34



DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD FOR WASTEWATER 260

Figure 35



DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD FOR WASTEWATER 280

DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD
FOR WASTEWATER 290

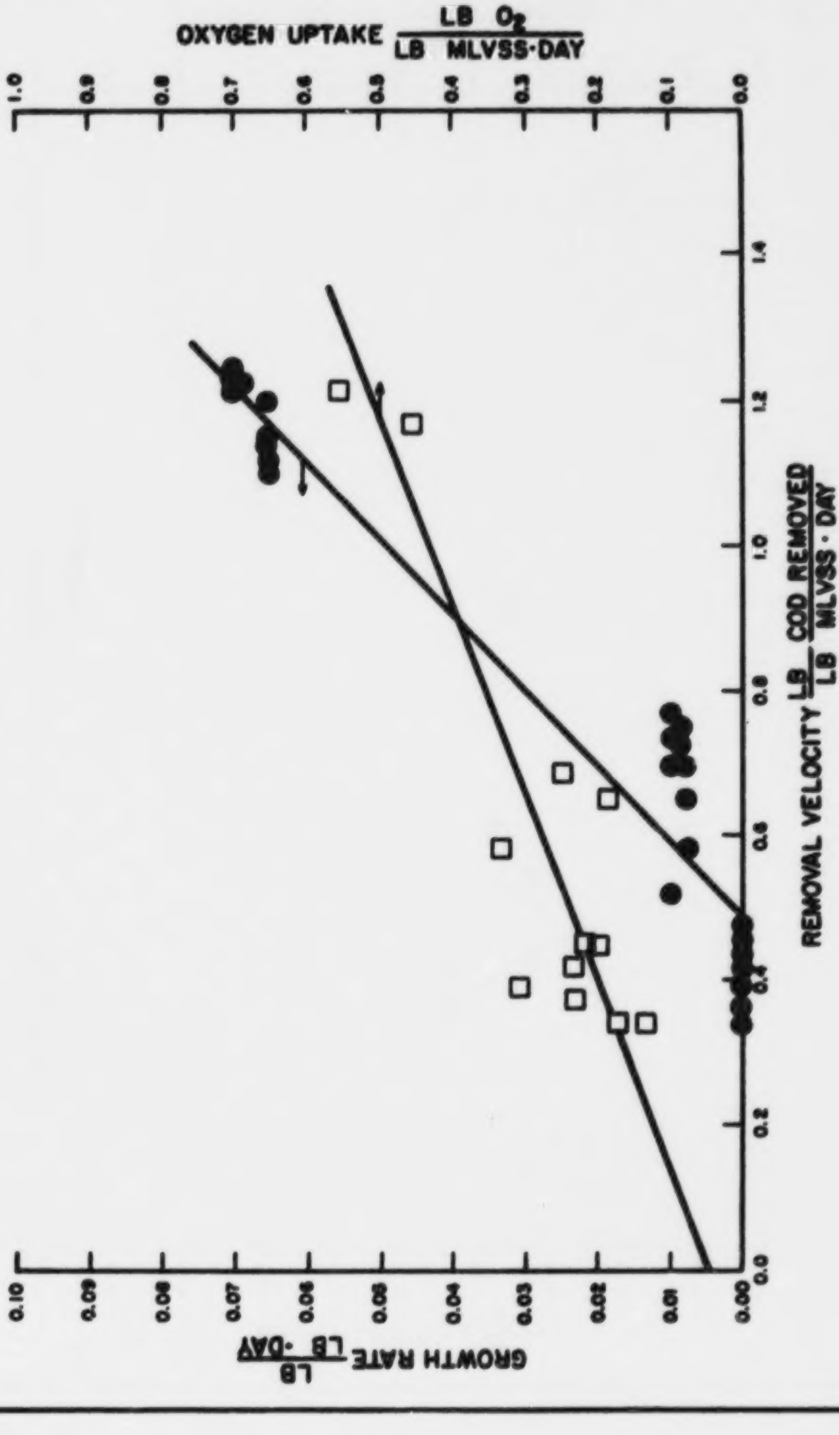


Figure 37

DETERMINATION OF KINETIC COEFFICIENTS BASED ON COD
FOR WASTEWATER 300

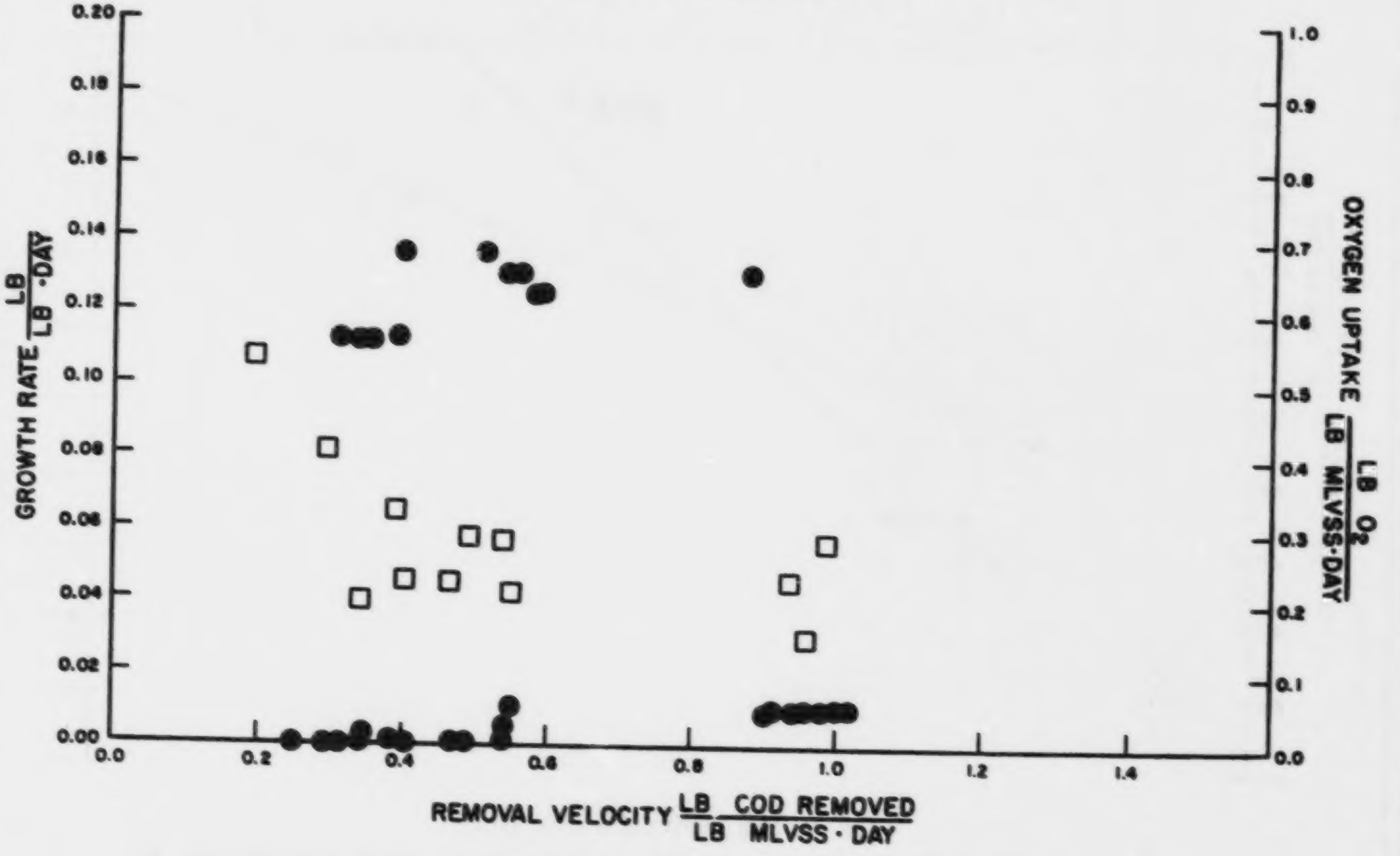


TABLE 19

SUMMARY OF KINETIC COEFFICIENTS
(BOD$_5$ BASIS)

	a	b	a'	b'
Wastewater	lbs sludge / lbs BOD$_5$ removed	lbs sludge oxidized / lbs sludge day	lbs oxygen req'd / lbs BOD$_5$ removed	lbs oxygen req'd / lbs sludge oxidized day
510	0.19	0.06	0.63	0.06
210	*	*	*	*
220	0.18	0.025	0.75	0.13
230	0	0	*	*
240	0.02	0.003	0.67	0.06
260	0.05	0.01	0.28	0.11
280	0.08	0.015	*	*
290	0.05	0.01	0.37	0.13
300	*	*	*	*

*Data did not fit a straight line

SUMMARY OF KINETIC COEFFICIENTS
(COD BASIS)

	a	b	a'	b'
Wastewater	lbs sludge / lbs COD removed	lbs sludge oxidized / lbs sludge day	lbs oxygen req'd / lbs COD removed	lbs oxygen req'd / lbs sludge oxidized day
510	0.06	0.025	0.25	0.13
210	0.00	0.00	*	*
220	*	*	*	*
230	0	0	*	*
240	0.006	0.002	0.33	0.04
260	0.04	0.07	*	*
280	0.004	0.001	*	*
290	0.10	0.045	0.40	0.05
300	*	*	*	*

*Data did not fit a straight line.

determined .

Summary

(1) The main conclusion to be drawn from the treatability studies is that all wastewaters investigated had BOD_5 removal (based on filtered effluent samples) in excess of 90 percent at loadings of approximately 0.20 pounds BOD_5/pound MLVSS/day. Wastewaters 240, 260, 290, and 300 continued to have BOD_5 removals in excess of 90 percent at loadings up to 0.60 pounds BOD_5/pound MLVSS/day. The results for the integrated wastewater indicated BOD_5 removals in excess of 95 percent at a loading of approximately 0.25 pounds BOD_5/pound MLVSS/day and removals above 90 percent at loadings up to 0.70 pounds BOD_5/pound MLVSS/day. The predicted effluent quality based on bench scale tests is presented in Section VIII, Table 54.

(2) Using the results for the treatability study for the integrated wastewater based on filtered effluent data, the following parameters would be applicable to the theoretical design of a regional plant.

Assume:
Influent soluble BOD_5 = 300 mg/l
Effluent soluble BOD_5 = 15 mg/l
MLVSS= 2000 mg/l
Loading = 0.50 lbs BOD_5/lb MLVSS/day
Removal velocity = 0.475 lbs BOD_5/lb MLVSS/day

Required:
Detention time = 0.3 days (no scale-up applied)
Oxygen required = 1,800 lbs/day/MGD (oxygen basis only)
Volatile sludge produced = 300 - 600 lbs/day/MGD
Effluent L_a = 36 mg/l
= 310 lbs/MGD

These results are only approximate and were modified as required based on subsequently obtained pilot plant results.

(3) The treatability studies indicated that with the possible exception of color and bioassay requirements, the activated sludge process could be used to treat the industrial wastewaters involved in the study to the quality level tentatively proposed by the DRBC. The true color of the industrial wastewaters, particularly the integrated wastewater, was not reduced significantly by biological treatment. Although the concentration of MBAS in the integrated wastewater effluent exceeded 10 mg/l, the data indicated that the high concentration was the result of interferences rather than detergents. Also, phenol removals for the integrated wastewater were in excess of 90 percent and resulted in an effluent concentration of approximately 0.30 mg/l.

Oxygen Transfer Studies

In this study the oxygen transfer parameters were determined for the integrated wastewater using both diffused air and mechanical aeration methods. The settled effluent from the bench scale reactor treating the integrated wastewater was chosen for analysis because its characteristics more closely resemble the fluid in an aeration basin than would the raw waste. It was decided not to conduct oxygen transfer experiments using the mixed liquor from the reactor because of the difficulty in establishing a true oxygen uptake by the activated sludge organisms.

These results are based on bench scale studies as described below. Subsequent analyses were conducted in the pilot plant operation using an "in situ" approach as described in Section VI.

Procedure

1. The aeration vessel was filled with six liters of tap water and the temperature recorded.

2. The solution was deoxygenated by the addition of a sodium sulfite solution containing a cobalt chloride catalyst.

3. The liquid was reaerated, measuring the dissolved oxygen concentrations at various time intervals. Reaeration was achieved using both sparged compressed air and a bench scale mechanical aerator.

4. The oxygen deficit versus time was plotted on semi-log paper.

5. The coeficients K_La, α, and B were calculated based on the following equations:

$$\frac{dc}{dt} = K_La\,(C_s - C) \qquad \text{(V-10)}$$

$$K_La = \frac{1}{T}\,\ln\,(C_s - C_o)\,/\,(C_s - C_t) \qquad \text{(V-11)}$$

$$B \quad = C_s\,(\text{Waste})\,/\,C_s\,(\text{Water}) \qquad \text{(V-12)}$$

$$\alpha \quad = K_La\,(\text{Waste})\,/\,K_La\,(\text{Water}) \qquad \text{(V-13)}$$

where:

$\frac{dc}{dt}$ = Rate of change of the dissolved oxygen concentration.

K_La = Overall oxygen transfer coefficient, $(\text{hour})^{-1}$

T = Time of aeration, hour.

C_s = Saturation concentration of oxygen in liquid, mg/l.

C_o = Concentration of oxygen in liquid at T = 0, mg/l

C_t = Concentration of oxygen in liquid at time T, mg/l

6. Steps 1 - 5 were repeated using an equal volume of settled effluent from the reactor treating the integrated wastewater.

7. Steps 1 - 6 were repeated using mechanical aeration equipment.

Results

The results of the oxygen transfer studies and the calculated coefficients are summarized in Table 20. The plots from which the determinations were made are shown in Figures 38 through 43. These include both the diffused and mechanical aeration tests.

As noted in Table 20, the oxygen transfer coefficient, α, decreased with an increase in organic loading for the diffused air studies. This is to be expected as more dissolved organic constituents are present in the effluent at the higher loading, and this will tend to reduce the oxygen transfer from the gas phase to the liquid phase across the liquid film. However, the K_La and α values derived from the mechanical aeration studies were rather erratic and it is recommended that these values be discarded as confirmatory pilot plant tests were conducted.

Zone Settling Analyses

Settling analyses were conducted on the mixed liquor from each of the bench scale reactors. For the units treating the individual wastewaters, the settling analyses were performed basically to determine the relative settleability of the individual sludges.

Data for the unit treating the integrated wastewater were further analyzed to

TABLE 20

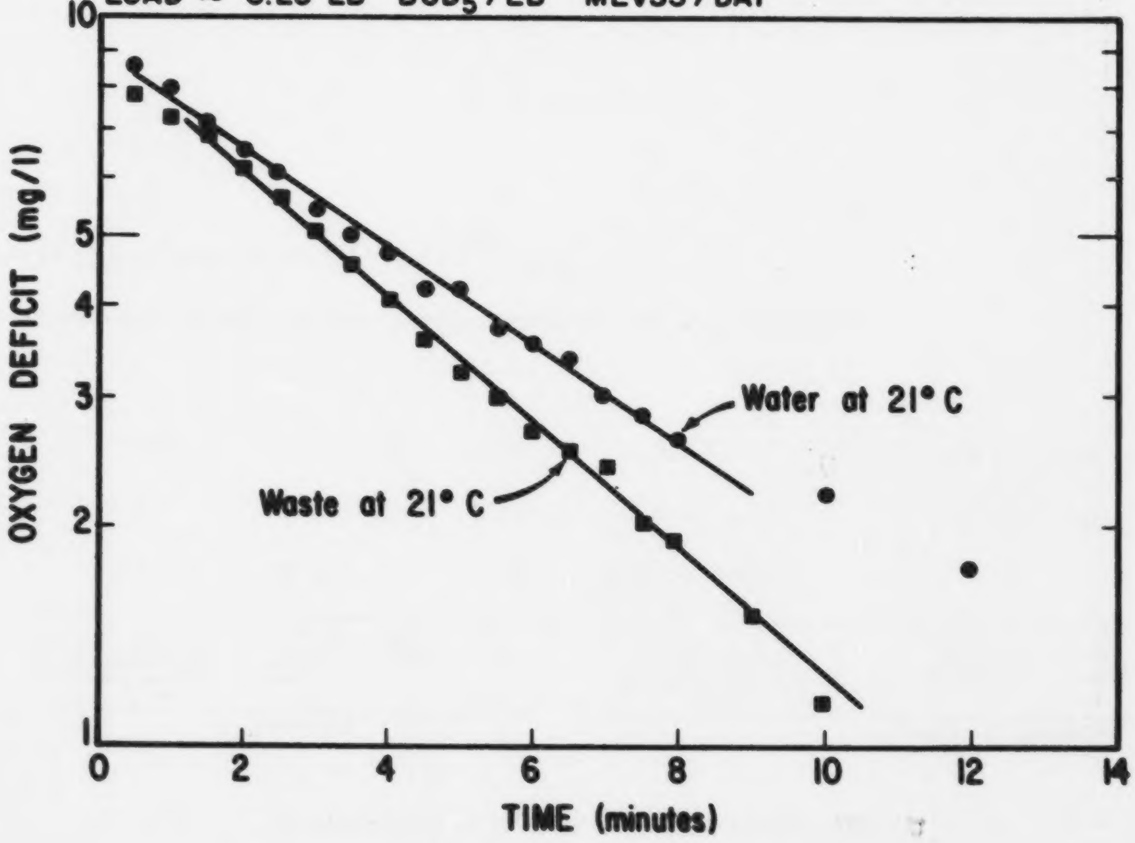
SUMMARY OF OXYGEN TRANSFER PARAMETERS

Bio-Reactor Loading $\frac{\text{lb BOD}_5}{\text{lb MLVSS/Day}}$	Diffused Air				Mechanical Aeration			
	K_La Wastewater (a) hr^{-1}	K_La Water hr^{-1}	Temp °C	α	K_La Wastewater (a) hr^{-1}	K_La Water hr^{-1}	Temp °C	α
~0.25	9.4	12.0	21	1.27	Data Inconsistent			
~0.50	6.5	8.5	21	0.92	4.4	2.8	21	1.55
~1.00	3.2(b)	5.2	25	0.61	7.1(b)	3.2	25	2.20

(a) Experiments conducted on the effluents from the Bio-Reactors at the various loadings.

(b) K_La corrected to 25°C by formula $K_La(T_1) = K_La(T_2)\ 1.028^{(T_1 - T_2)}$

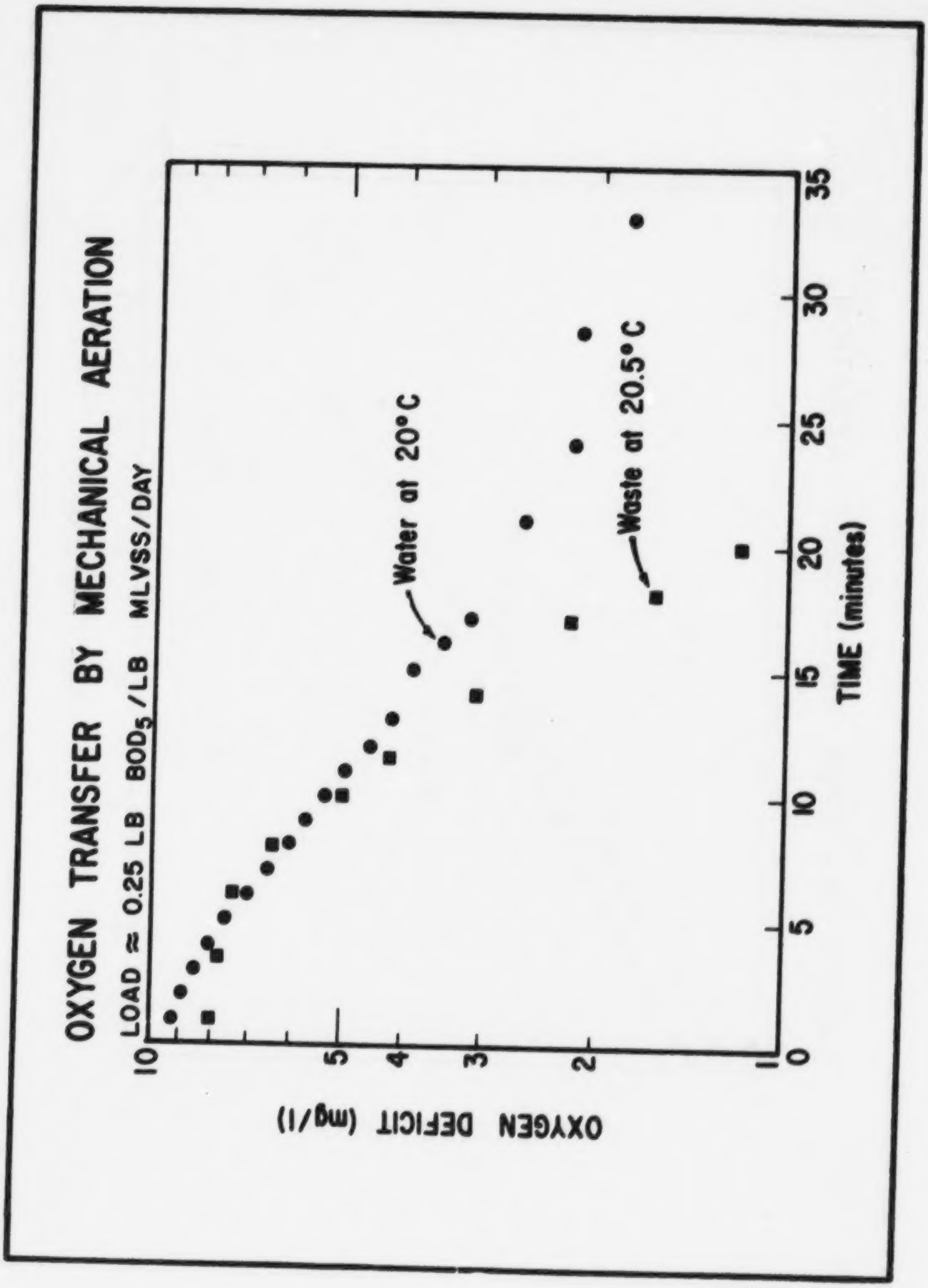
Figure 38



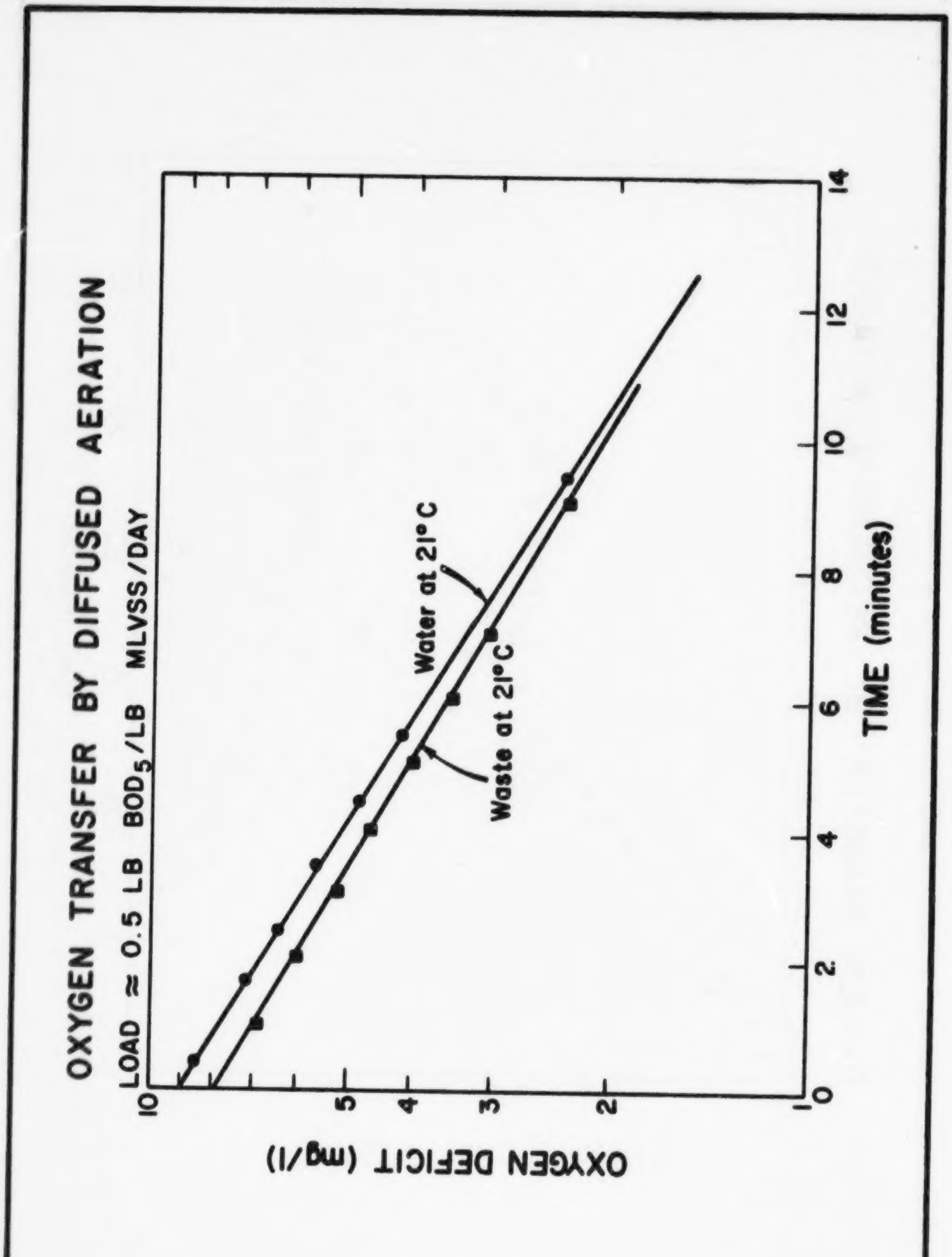
OXYGEN TRANSFER BY DIFFUSED AERATION

LOAD ≈ 0.25 LB BOD_5/LB MLVSS/DAY

Figure 39



OXYGEN TRANSFER BY MECHANICAL AERATION



OXYGEN TRANSFER BY DIFFUSED AERATION

LOAD ≈ 0.5 LB BOD_5/LB MLVSS/DAY

Figure 41

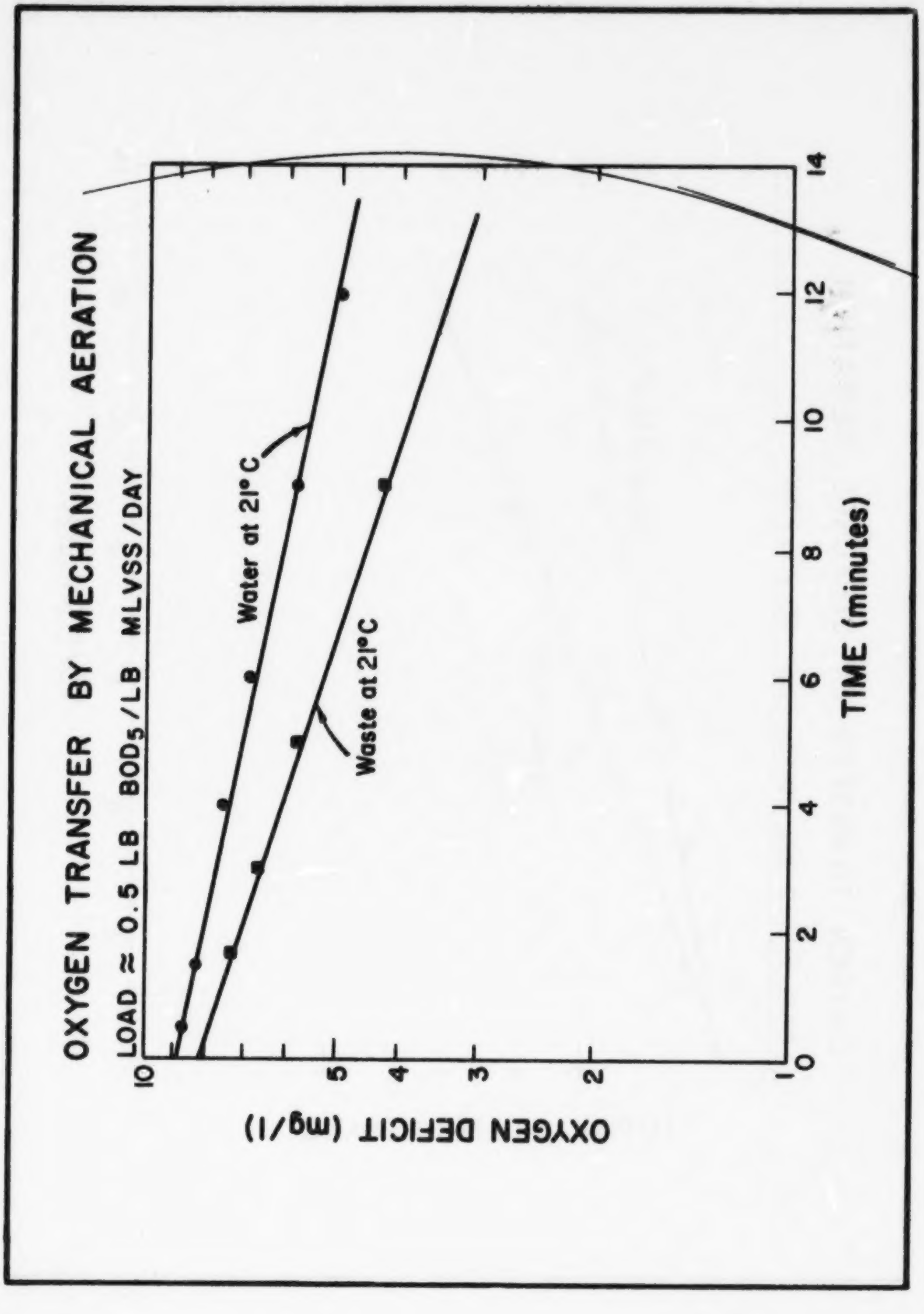
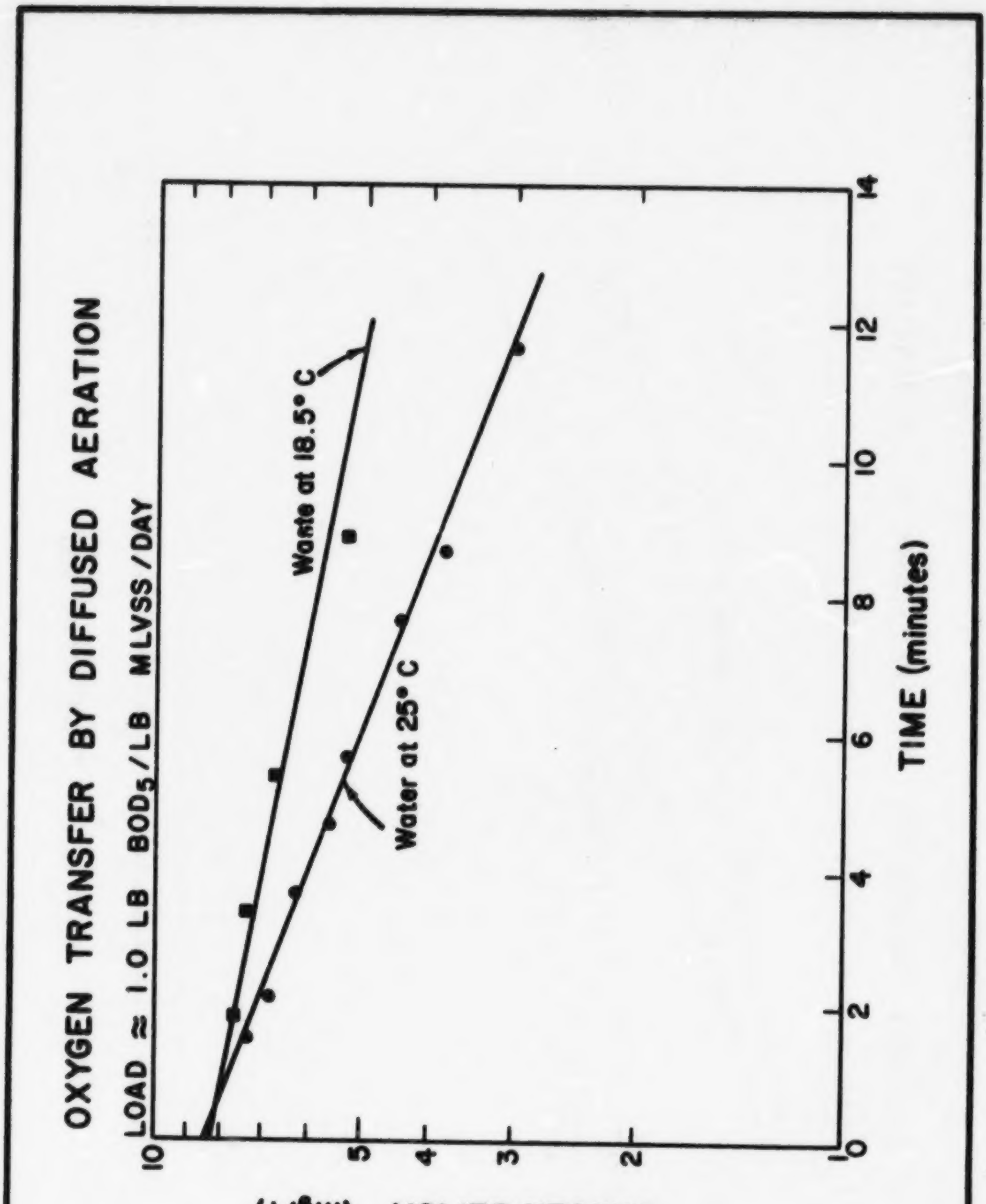

OXYGEN TRANSFER BY MECHANICAL AERATION
LOAD ≈ 0.5 LB BOD$_5$/LB MLVSS/DAY
Water at 21°C
Waste at 21°C
OXYGEN DEFICIT (mg/l)
10
5
4
3
2
1
TIME (minutes)
0
2
4
6
8
10
12
14

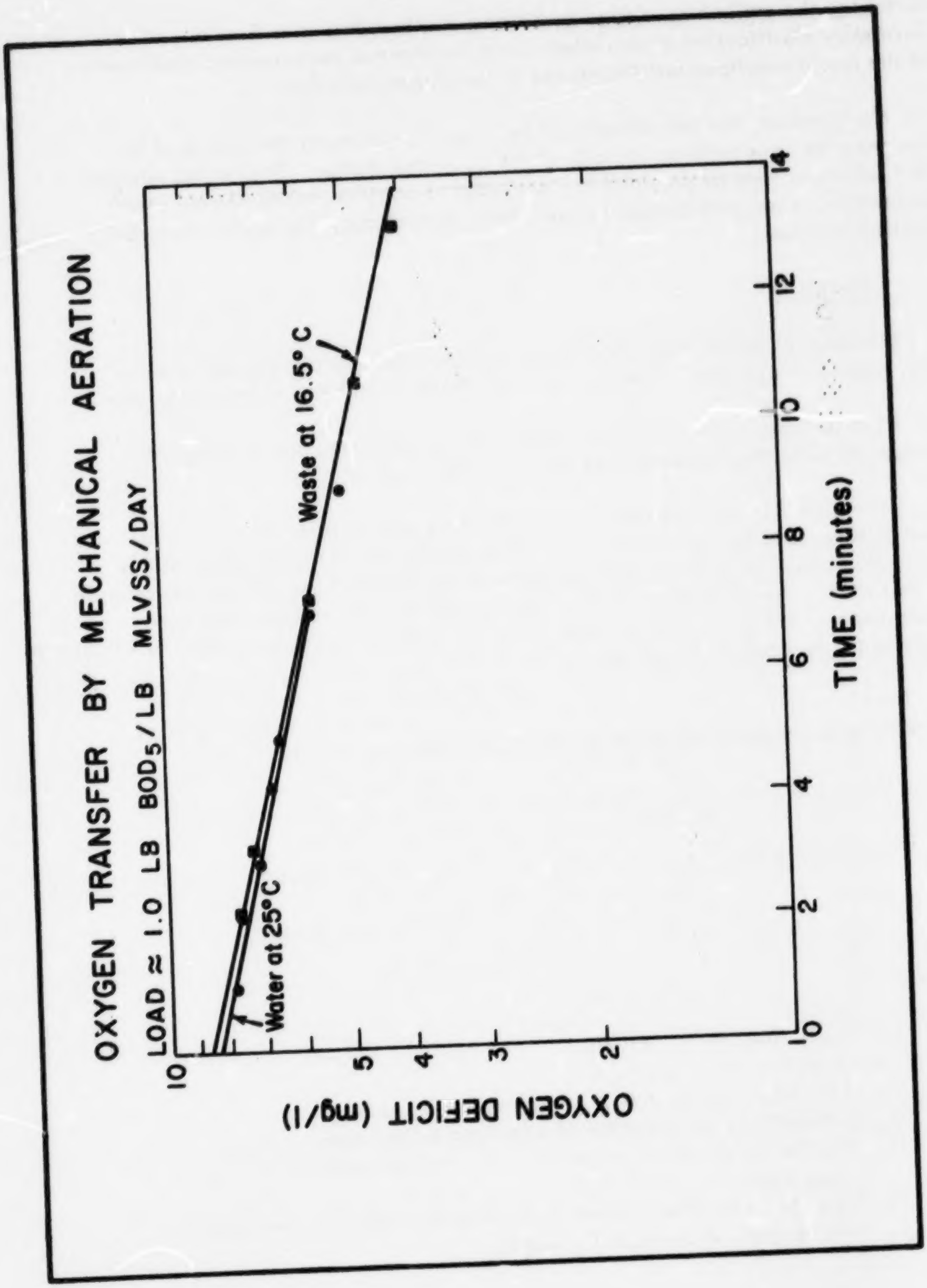
Figure 42



OXYGEN TRANSFER BY DIFFUSED AERATION

LOAD ≈ 1.0 LB BOD$_5$/LB MLVSS/DAY

Figure 43


OXYGEN TRANSFER BY MECHANICAL AERATION
LOAD ≈ 1.0 LB BOD$_5$/LB MLVSS/DAY
Waste at 16.5° C
Water at 25°C
OXYGEN DEFICIT (mg/l)
10
5
4
3
2
1
TIME (minutes)
0
2
4
6
8
10
12
14

determine the preliminary design parameters for secondary clarification. Secondary clarification of activated sludge involves two requirements: clarification of the liquid overflow; and thickening of the sludge underflow.

For clarification, the rise velocity of the liquid overflowing the tank must be less than the zone settling velocity of the activated sludge. Thickening requires that sufficient time be provided for the sludge to compress to the desired concentration. Both criteria must be considered in analyzing the results of sludge settling analyses.

Procedure

1. One liter of mixed liquor from each biological reactor was placed in a one liter graduated cylinder. Samples were also taken for suspended solids analyses.

2. Zone settling curves were then determined by measuring and plotting the sludge interface height versus time for each individual unit.

3. The results for the integrated wastewater were converted to a plot of interface settling velocity versus the solids concentration by taking the slope of the curve from step 2 at various times and calculating the resulting solids concentration at that time. The allowable overflow rate in gpd/sq ft for various inlet concentrations of solids can then be determined by multiplying the zone settling velocity in ft/hr by (24 hr/day) (7.48 gal/cu ft).

4. The allowable overflow rate for the integrated wastewater based on sludge thickening was determined by the equations presented as follows:

$$UA = \frac{T_u}{C_o H_o} \times \frac{10^6}{62.4} \text{ lb/cu ft} \qquad \text{(V-14)}$$

$$C_o H_o = C_u H_u \qquad \text{(V-15)}$$

$$OR = \frac{1}{UA\, C_o} \times \frac{C_u - C_o}{C_u} \times \frac{10^6}{8.33} \text{ lb/gal} \qquad \text{(V-16)}$$

where:

UA = unit area, sq ft - day/lb
OR = overflow rate, gpd/sq ft
C_o = initial concentration of suspended solids, ppm
C_u = underflow concentration of suspended solids, ppm
H_o = initial height of the mixed liquor in the graduated cylinder - 1.15 ft (13.8 in).
H_u = height of the sludge layer at the desired underflow concentration
T_u = time required to reach C_u and H_u

Results

The zone settling curves from the individual industrial biological reactors at loadings of 0.25, 0.50, and 1.0 lbs BOD_5/lb MLVSS/day are presented in Figures 44, 45, and 46 respectively. These data indicate good settling of sludges based on the batch sludge settling approach. It should be recognized, however, that prototype clarifiers exhibit different characteristics than what might be observed in a graduated cylinder. However, these results do indicate good biological solids - liquid separation and offer some basis for estimating the design overflow rates.

The zone settling curves for the biological reactors treating the integrated industrial wastewater at each of the three loadings are summarized in Figure 47. These results have been further analyzed by taking the slope of the curves at various times and calculating the resulting solids concentration in order to depict the interface settling velocity versus solids concentration as shown in Figure 48. The design parameters for the integrated wastewater based on thickening the underflow to a concentration of 10,000 mg/l are tabulated in Table 21. Based on these data, an overflow rate of 1,600 gpd/ft^2 would be permissible at a design organic loading of 0.5 lbs BOD_5/lb MLVSS/day. However, lower overflow rates should probably be considered based on past experience relative to scale-up.

ANCILLARY BENCH SCALE STUDIES

Bacterial Quality Characterization

In order to determine the need for disinfection, coliform determinations were made on the raw industrial wastewaters and on the effluents from the bench scale reactors.

Coliform organisms can result from both fecal and non-fecal sources. Both types of organisms were investigated. Hereafter, the designation "coliforms" includes all coliforms whether fecal or non-fecal, and "fecal coliforms" refers only to those organisms that are primarily the result of fecal contamination.

The DRBC standards require disinfection of any wastewater having an average fecal coliform concentration in excess of 200 organisms per 100 milliliters. Because of the low pH and limited contamination of the industrial wastewaters, it is probable that fecal organisms would be sufficiently destroyed to preclude the need for chlorination or other means of disinfection.

Municipal sewage was not investigated in this task as coliform counts for individual sewage effluents are well-documented and such data would have little meaning.

Figure 44

ZONE SETTLING CURVES FOR INDIVIDUAL WASTEWATERS

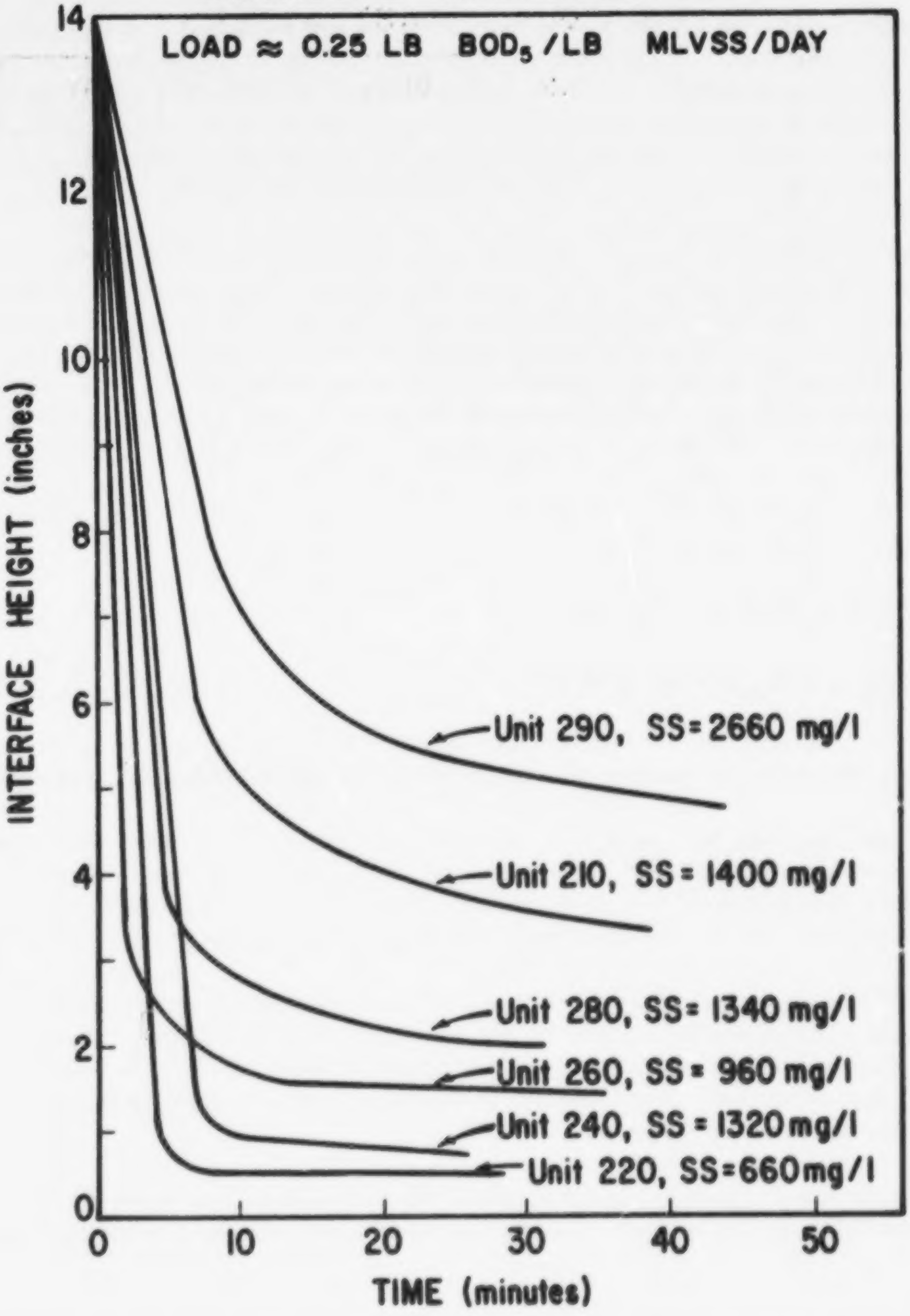


Figure 45

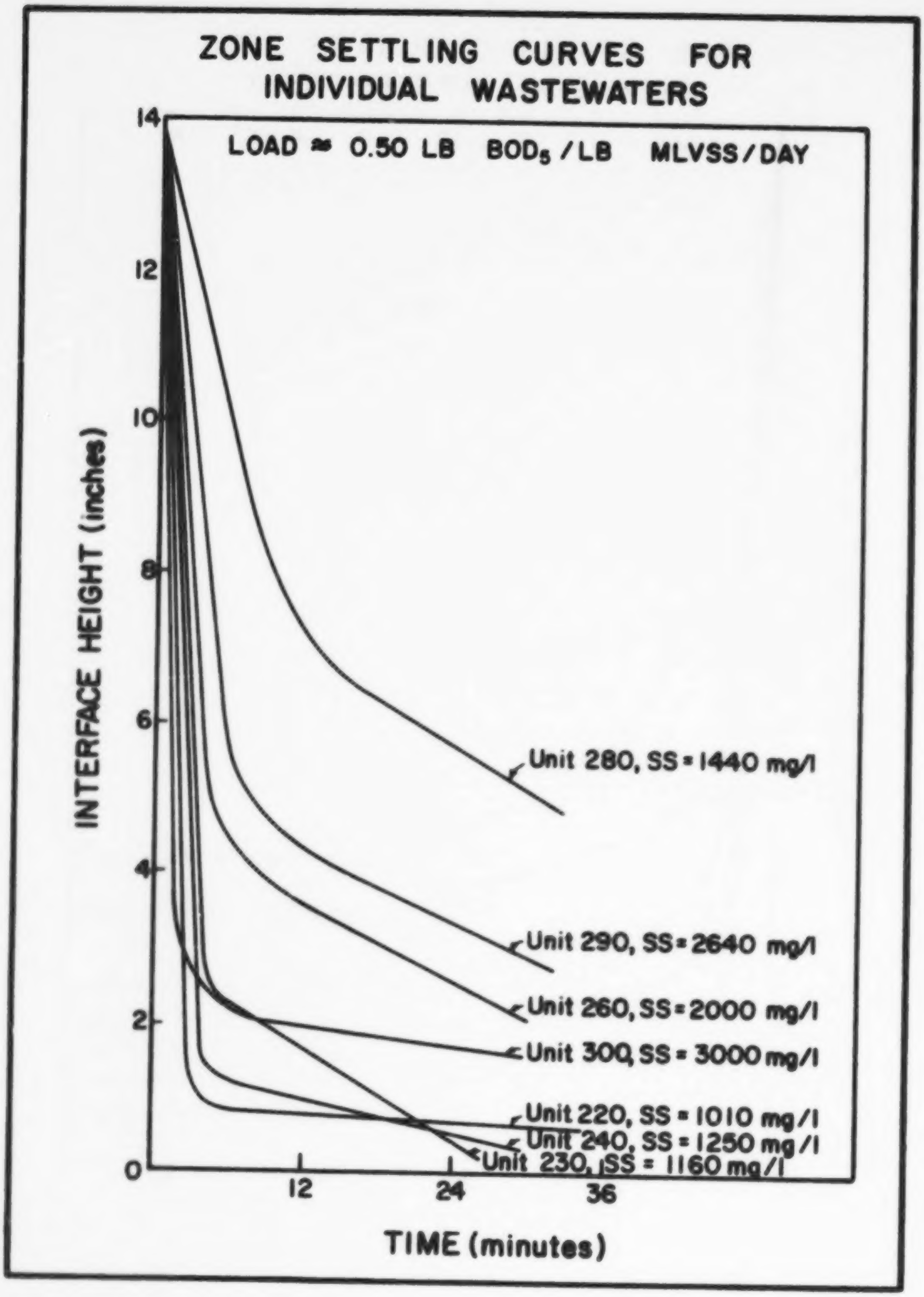

ZONE SETTLING CURVES FOR INDIVIDUAL WASTEWATERS
LOAD ≈ 0.50 LB BOD5 /LB MLVSS/DAY
INTERFACE HEIGHT (inches)
14
12
10
8
6
4
2
0
12
24
36
TIME (minutes)
Unit 280, SS = 1440 mg/l
Unit 290, SS = 2640 mg/l
Unit 260, SS = 2000 mg/l
Unit 300, SS = 3000 mg/l
Unit 220, SS = 1010 mg/l
Unit 240, SS = 1250 mg/l
Unit 230, SS = 1160 mg/l

Figure 46

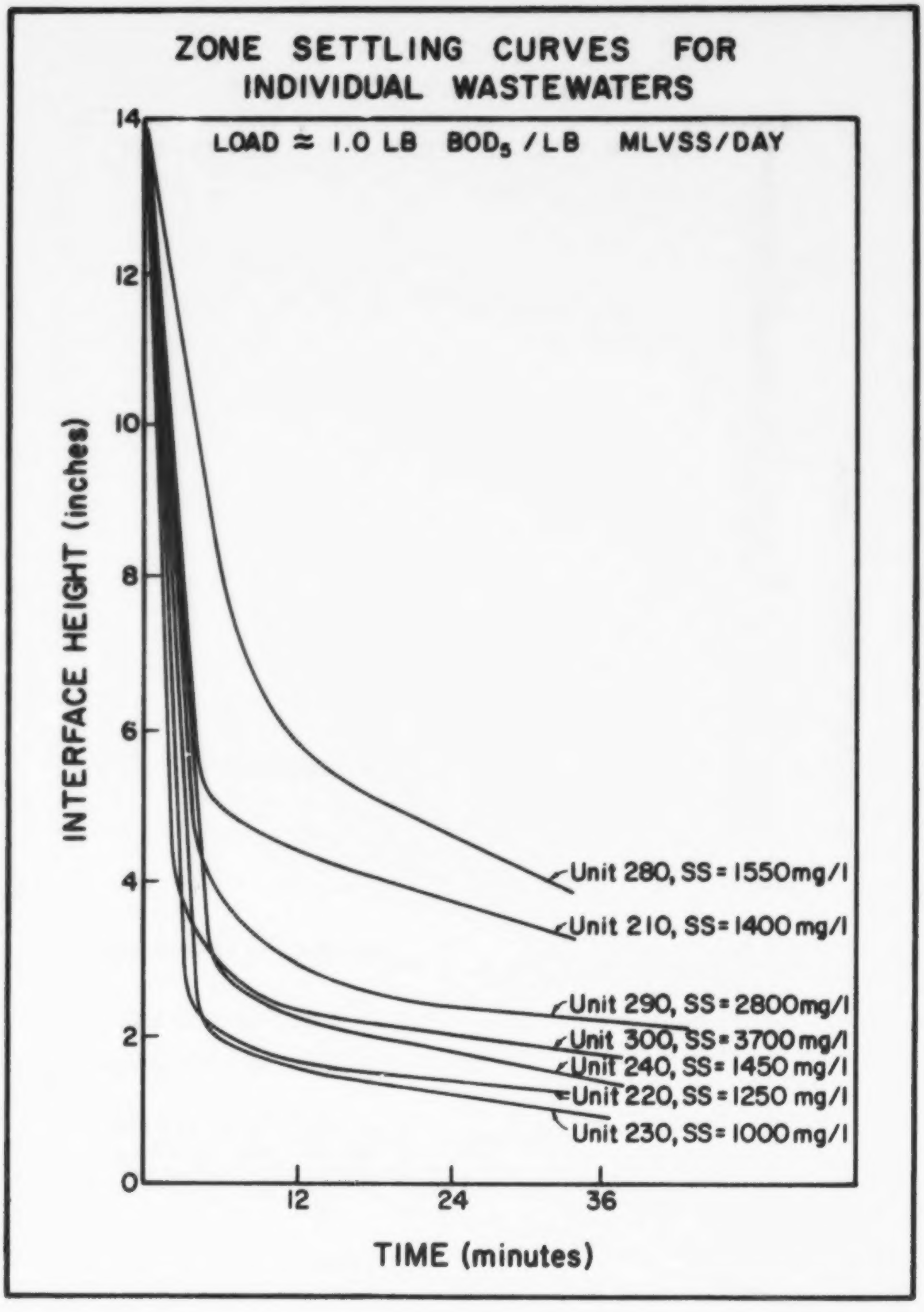

ZONE SETTLING CURVES FOR INDIVIDUAL WASTEWATERS
LOAD ≈ 1.0 LB BOD₅ / LB MLVSS/DAY
INTERFACE HEIGHT (inches)
TIME (minutes)
0
2
4
6
8
10
12
14
12
24
36
Unit 280, SS = 1550 mg/l
Unit 210, SS = 1400 mg/l
Unit 290, SS = 2800 mg/l
Unit 300, SS = 3700 mg/l
Unit 240, SS = 1450 mg/l
Unit 220, SS = 1250 mg/l
Unit 230, SS = 1000 mg/l

Figure 47

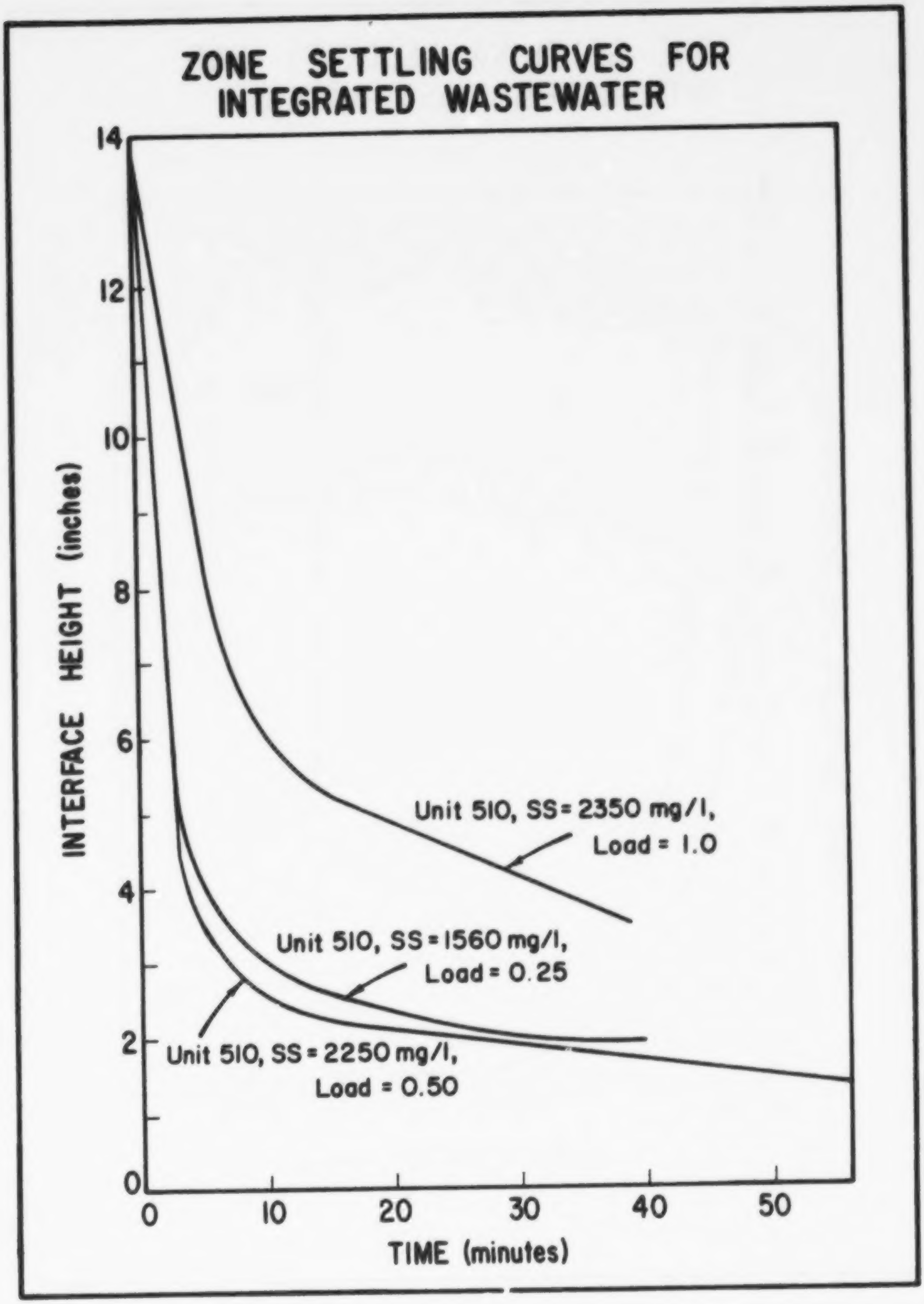

ZONE SETTLING CURVES FOR INTEGRATED WASTEWATER
INTERFACE HEIGHT (inches)
TIME (minutes)
0
2
4
6
8
10
12
14
0
10
20
30
40
50
Unit 510, SS = 2350 mg/l, Load = 1.0
Unit 510, SS = 1560 mg/l, Load = 0.25
Unit 510, SS = 2250 mg/l, Load = 0.50

Figure 48

ZONE SETTLING VELOCITY FOR INTEGRATED WASTEWATER

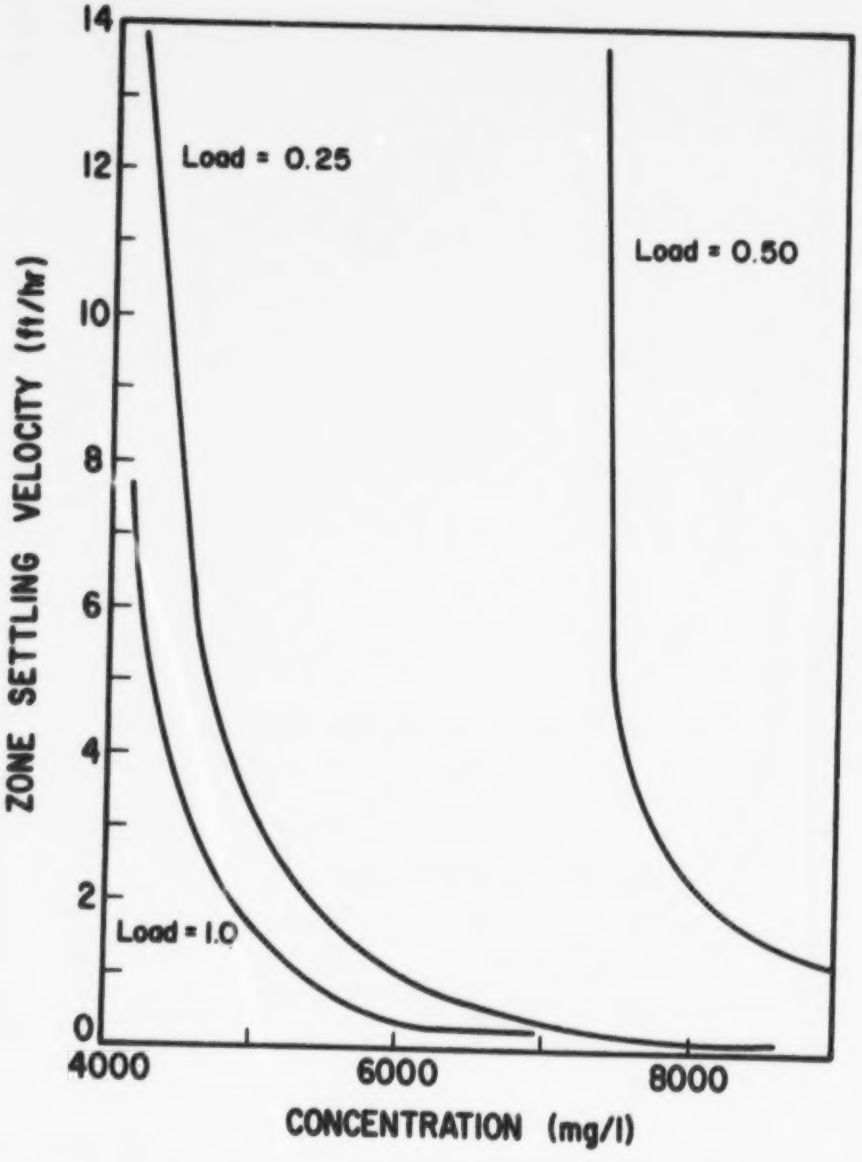


TABLE 21

MIXED LIQUOR THICKENING RESULTS FOR THE INTEGRATED WASTEWATER

Loading lbs BOD_5 lbs MLVSS/day	Suspended Solids Assumed Underflow mg/l	Suspended Solids Influent mg/l	Unit Area sq ft - day/lb	Overflow Rate gpd/sq. ft
~0.25	10,000	1,560	0.16	400
~0.50	10,000	2,250	0.0259	1,600
~1.0	10,000	2,350	0.173	230

Procedure

Samples of the raw wastewaters and of the effluents from the biological reactors at loadings of 0.50 and 1.0 lbs BOD_5/lb MLVSS/day were tested for the presence of coliform organisms and fecal coliforms using the Millipore Filter Technique as described in Standard Methods.

Results

The results of all determinations are summarized in Table 22.

Coliforms were found in only two of the raw wastewaters. Wastewater 021 had 1,300 coliforms per 100 milliliters, but only 30 were of the fecal group. Wastewater 31 had 10 coliforms per 100 milliliters, but none were of the fecal origin.

All but one of the bench scale reactors had coliforms in their effluents. None of these, however, were of the fecal group.

Summary

Based on a limited number of samples, fecal organisms in the raw industrial wastewaters appear to be sufficiently destroyed to not require disinfection.

Coliform organisms do appear in the effluents from the reactors. The organisms probably were a result of the initial seeding of the reactors, which was done with an activated sludge treated municipal sewage. It would appear that the coliforms are now an active part of the bacterial population and would not require disinfection because they are not of fecal origin.

Chlorination Evaluation

Chlorine demand tests were performed on the effluents from each of the reactors to determine how much chlorine each of the individual wastewaters would require to meet the Delaware River Basin Commission's standards for disinfection. These standards call for a residual of 1.0 mg/l free chlorine after a contact time of 15 minutes. The standards do not mention a combined chlorine residual and, therefore, these evaluations were limited to free chlorine.

It was determined that the bacterial quality of the individual wastewaters was such that disinfection probably would not be required. However, depending upon the degree of contamination from municipal contributors, disinfection could become necessary and therefore the amount of chlorine required for each stream was determined.

TABLE 22

COLIFORM ORGANISMS IN INDUSTRIAL WASTEWATERS

Industry	Raw Waste		Bio-Reactor Effluents at Loadings of 0.5	1.0
	Fecal #/100 ml	Total #/100 ml	Total #/100 ml	Fecal #/100ml
011	0	0	80	0
021	30	1300	1860	0
031	0	10	180	0
041	0	0	60	0
061	0	0	0	0
071	0	0		
081	0	0	100	0
091	0	0	>2000	0
101	0	0	20	0
191	0	0	>2000	0

The orthotolidine flash method was chosen for the determination of free residual chlorine. Although it is a qualitative technique, it is sufficiently accurate for the purposes of this task. Other orthotolidine methods were not used because of potential interferences from nitrite nitrogen and color.

Procedure

1. One hundred milliliter portions of the effluent from each of the reactors at a loading of approximately 0.5 lbs BOD_5/lb MLVSS/day were placed in beakers and the color and odor observed.

2. The samples were dosed with varying amounts of a standard hypochlorite solution, agitated, and allowed to stand for 15 minutes.

3. After 15 minutes contact time, the free chlorine residual was determined in each sample using the orthotolidine flash method as described in Standard Methods. The effect on color and odor was also observed.

Results

All results are summarized in Table 23. The probable dose of the individual samples was taken as the average of the sample having a free residual and the sample not having a free residual.

The sum of the individual requirements is greater than that indicated for the integrated wastewater. This could be the result of interactions that are taking place to reduce the chlorine demand of the integrated sample, or it could be the result of experiment error.

No significant effect on odor or color was observed in any of the samples.

Summary

The results of this task indicate that approximately 25 to 30 lbs chlorine per MGD would be required to obtain a free chlorine residual of 1.0 mg/l after a 15 minute contact time.

Only one wastewater had an abnormally high chlorine demand. However, because of the low flow of this particular wastewater, it does not have significant effect on the integrated wastewater.

Because of nitrite and color interferences, the Amperometric Titration Method should be used to determine chlorine residuals if a high degree of accuracy is required.

TABLE 23

CHLORINE DEMAND OF INDUSTRIAL WASTEWATERS

Wastewater	Chlorine Dose (a) Highest Without Free Cl Residual mg/l	Chlorine Dose (a) Lowest With Free Cl Residual mg/l	Probable Dose mg/l	Flow MGD	Probable Chlorine Required lb/day (b)	Comments
010	60	70	65.0	0.14	76	Slight chlorine odor. Duplicate results at T = 5°C and T = 20°C.
020	1	3	2.0	5.4	90	No significant odor.
030	1	3	2.0	24.0	400	Slight chlorine odor.
040	5	10	7.5	38.6	2,420	Slight chlorine odor.
060	5	10	7.5	2.4	150	Slight chlorine odor.
080	1	5	3.0	3.0	75	No significant odor.
090	10	15	12.5	1.15	120	Color interference. Slight chlorine odor.
100	10	20	15.0	3.0	380	Slight chlorine odor.
510 (c)	1	5	3.0	77.7	1,940	Slight chlorine odor.

(a) All tests performed at 5°C except as noted.

(b) The sum of industries 10 through 100 equals 3,700 lb/day.

(c) Integrated wastewater.

Preliminary Activated Carbon Study

Adsorption is a process by which a substance (the adsorbate) is taken up and becomes attached to the surface of a solid (the adsorbent). The process is selective in all practical applications, and one component of a mixture may be adsorbed to a greater extent than another.

Adsorbents have found direct application in wastewater treatment for the removal of organic constituents which are difficult or impossible to remove by conventional biological treatment processes. The adsorbent which is most commonly applied to wastewaters is activated carbon.

In this study, effluent from the bench scale reactor treating the composite wastewater at a loading of approximately 1.0 lbs BOD_5/lbs MLVSS/day was treated with activated carbon to determine the effect on chemical oxygen demand (COD), biochemical oxygen demand (BOD), methylene blue active substances (MBAS), phenol, color, and odor. The reactor did not produce sufficient effluent to operate a continuous carbon column, and the investigation was therefore limited to batch studies. Subsequent batch and column studies were performed during the pilot plant phase of the project and this information is presented in Section VI.

Procedure

1. The activated carbon was soaked for 24 hours in distilled water, then oven-dried for 24 hours at 103°C.

2. Doses of 41, 68, and 200 mg of the powdered carbon were placed in test flasks and one liter portions of filtered effluent from the reactor treating the integrated wastewater were added.

3. Samples were taken every 15 minutes and filtered immediately. This was continued until the equilibrium concentration was obtained.

4. The COD, BOD, MBAS, phenol, color and odor of the raw and treated samples were measured.

Data Analysis

The Freundlich isotherm is commonly used to correlate batch adsorption data. The equation is based on empirical relationships and at equilibrium may be expressed as:

$$X/M = kC^{1/n} \qquad (V-17)$$

where:

X = the weight of the substance adsorbed
M = the weight of the adsorbent
C = the concentration remaining in solution
k and n = empirical constants depending on temperature, the adsorbent, and the substance to be adsorbed

Based on this formulation, X/M versus C should plot as a straight line on log paper thus facilitating both the determination of k and n, and the interpolation of data.

Results

The effect of activated carbon on COD, BOD, MBAS, and phenol are summarized in Table 24. The Freundlich isotherms for COD, MBAS, and phenol are presented in Figures 49 through 51, respectively.

The equilibrium concentration was reached in approximately 30 minutes for COD, BOD, and MBAS. Phenol equilibrium occurred after one hour, with the longer equilibrium period probably explained by the dilute initial concentration of phenol.

The results indicate that most of the dissolved BOD remaining after biological treatment can be removed with an activated carbon dose of less than 41 mg/l.

Extrapolation of the MBAS isotherm indicates that a dose of over 500 mg/l activated carbon would have been required to reduce the MBAS concentration to the DRBC river objective of 1.0 mg/l. However, interferences attributable to specific acids in the wastewater render this data questionable, and the results should be interpreted in this context. Similarly, to reduce the phenol concentration of the raw wastewater to 0.2 mg/l approximately 430 mg/l activated carbon would be required.

During testing, significant color reduction was observed at the 200 mg/l activated carbon dose, with the deep brown initial color diminishing to a very pale yellow. Indications were that a carbon dose slightly greater than 200 mg/l would remove most of the color-causative compounds.

The wastewater before activated carbon treatment did not have a noticeable odor and therefore no effect could be determined.

Summary

The results of the activated carbon batch studies indicate that most of the soluble BOD remaining after biological treatment was removed with an activated carbon

Figure 49

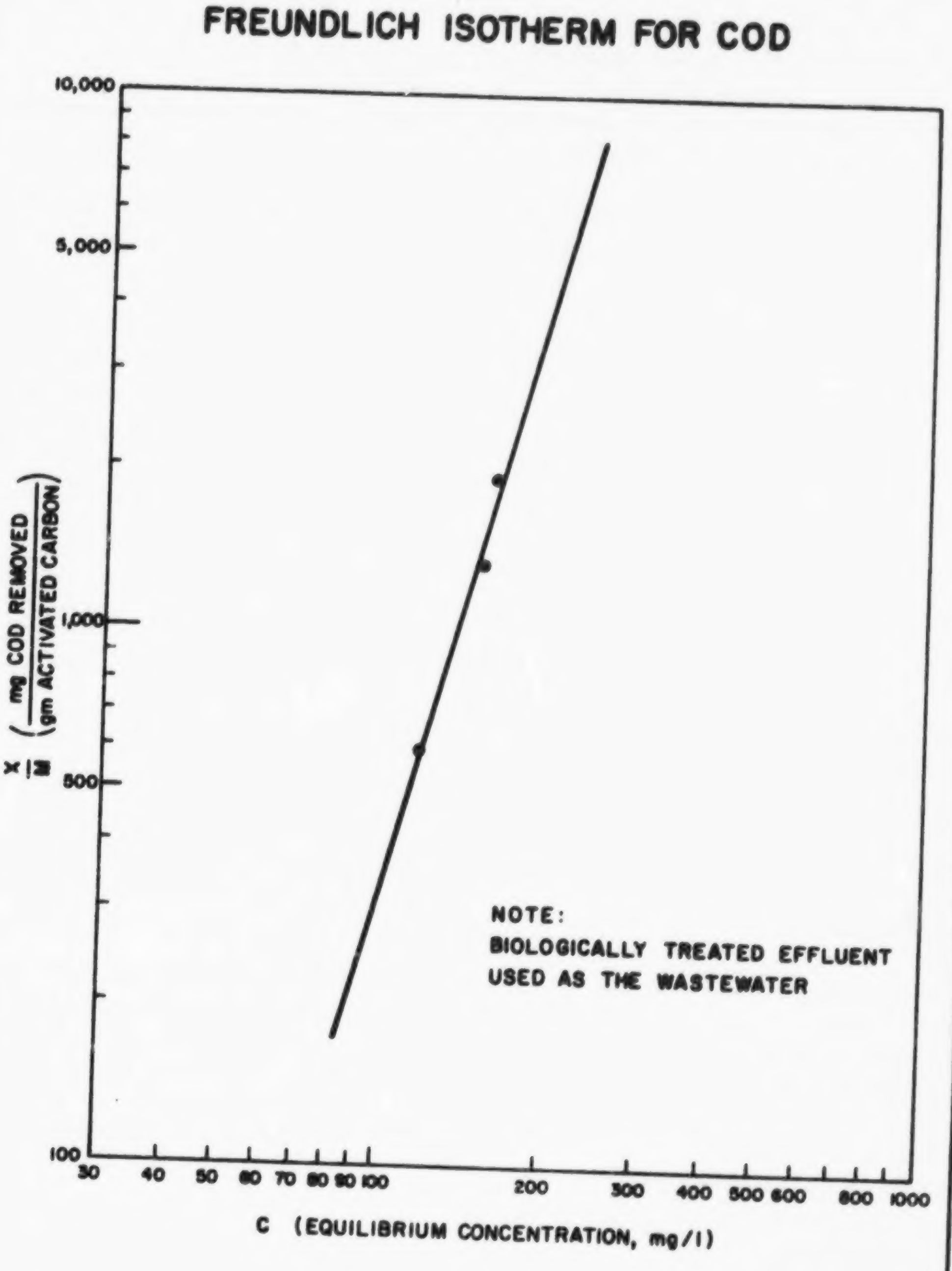

FREUNDLICH ISOTHERM FOR COD
10,000
5,000
1,000
500
100
30
40
50
60
70
80
90
100
200
300
400
500
600
800
1000
C (EQUILIBRIUM CONCENTRATION, mg/l)
x/m (mg COD REMOVED / gm ACTIVATED CARBON)
NOTE:
BIOLOGICALLY TREATED EFFLUENT
USED AS THE WASTEWATER

Figure 50

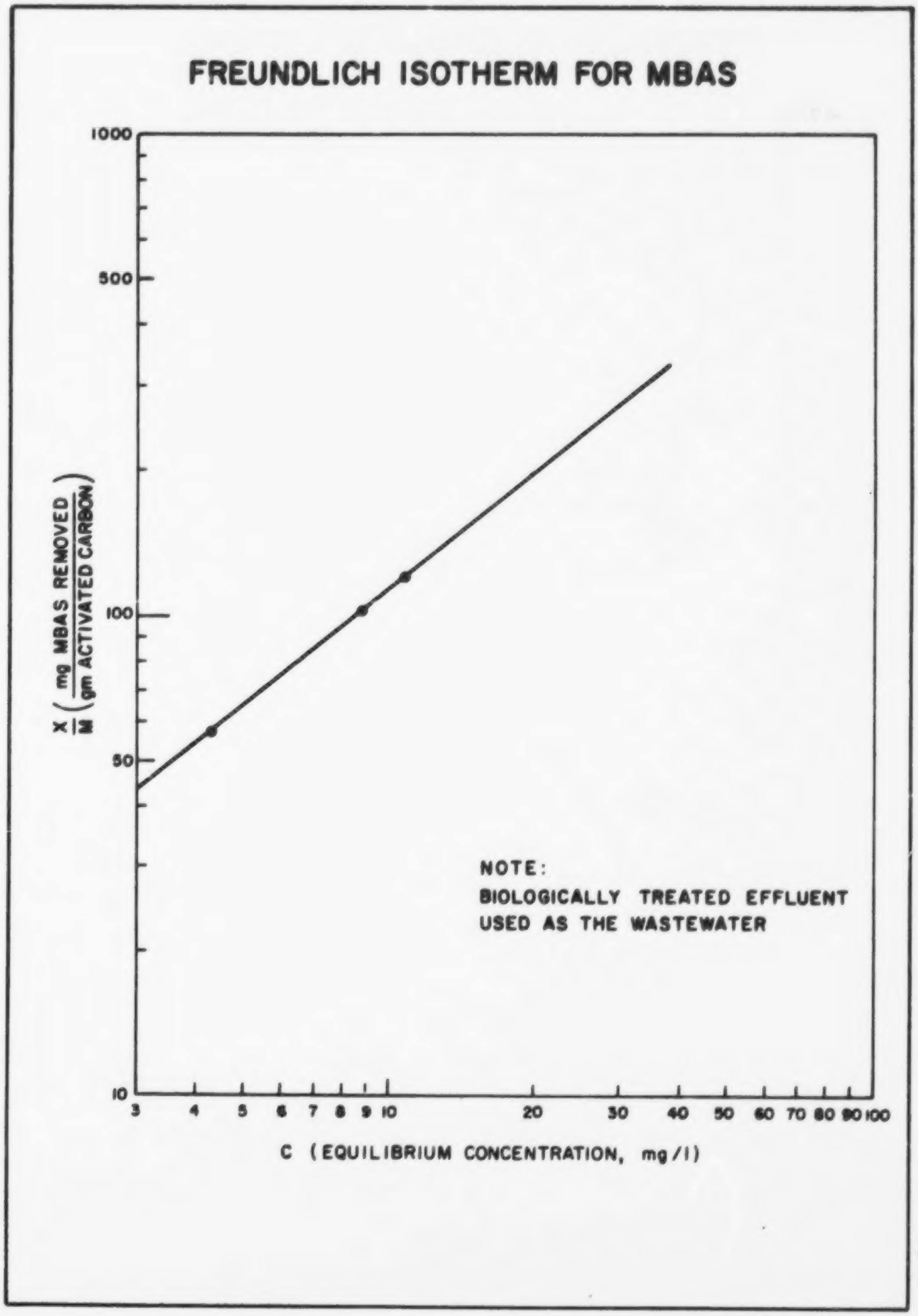

FREUNDLICH ISOTHERM FOR MBAS
1000
500
100
50
10
$\frac{X}{M}$ $\left(\frac{\text{mg MBAS REMOVED}}{\text{gm ACTIVATED CARBON}}\right)$
3
4
5
6
7
8
9
10
20
30
40
50
60
70
80
90
100
C (EQUILIBRIUM CONCENTRATION, mg/l)
NOTE:
BIOLOGICALLY TREATED EFFLUENT
USED AS THE WASTEWATER

Figure 51

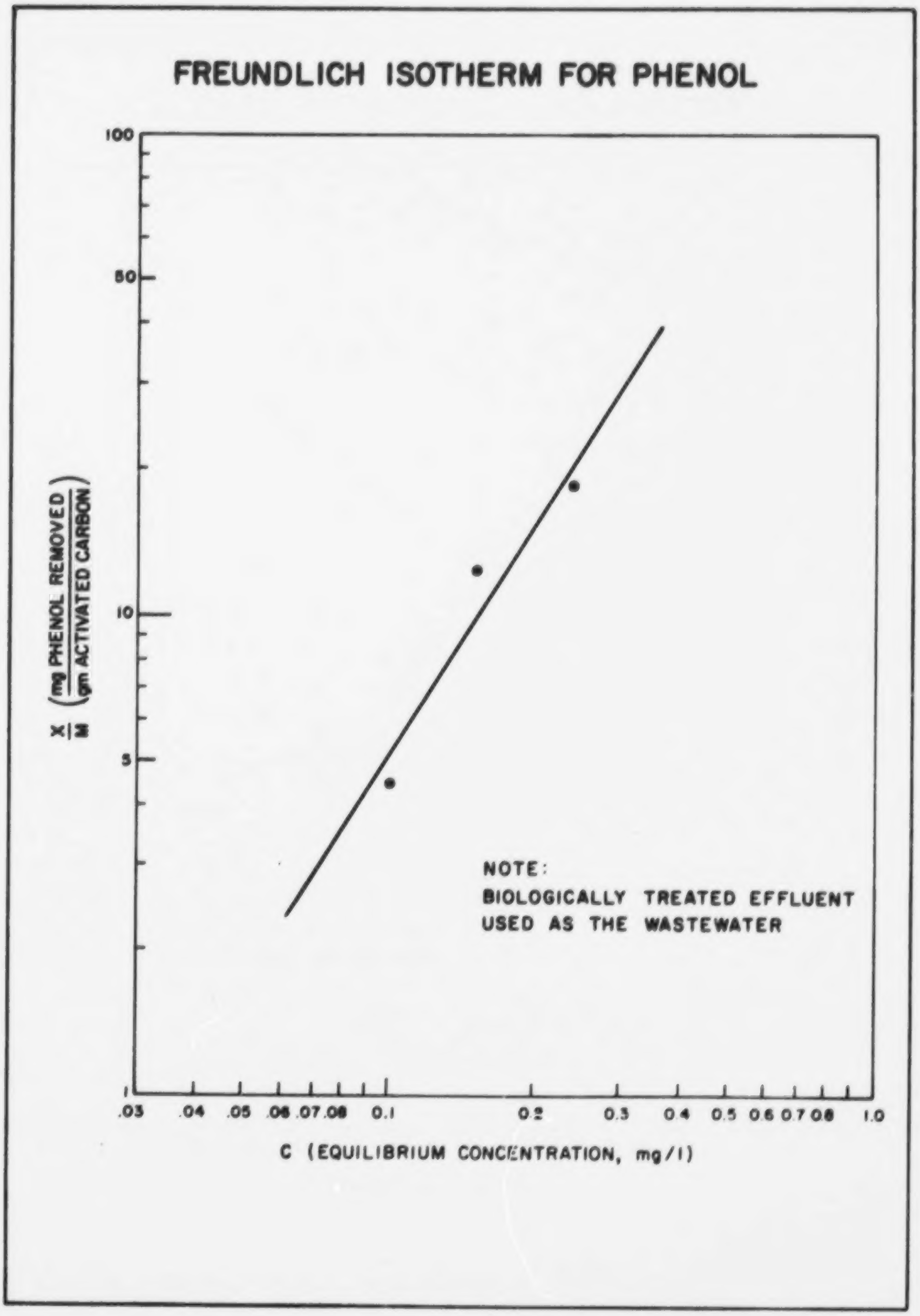

FREUNDLICH ISOTHERM FOR PHENOL
100
50
10
5
1
X/M (mg PHENOL REMOVED / gm ACTIVATED CARBON)
.03
.04
.05
.06
.07
.08
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
1.0
C (EQUILIBRIUM CONCENTRATION, mg/l)
NOTE:
BIOLOGICALLY TREATED EFFLUENT
USED AS THE WASTEWATER

TABLE 24

SUMMARY OF RESULTS FOR ACTIVATED CARBON BATCH STUDY

	COD			BOD$_5$			MBAS			PHENOL		
	Concentration		Percent	Concentration		Percent	Concentration		Percent	Concentration		Percent
Carbon Dose[a] mg/l	Initial mg/l	Equil. mg/l	Removal	Initial mg/l	Equil. mg/l	Removal	Initial mg/l	Equil. mg/l	Removal	Initial mg/l	Equil. mg/l	Removal
41	235	156	33.6	67	1.6	97.6	15.7	10.8	31.2	1.0	0.24	76.0
68	235	145	38.3	67	1.6	97.6	15.7	8.8	44.0	1.0	0.15	85.0
200	235	116	50.6	67	1.6	97.6	15.7	4.3	72.6	1.0	0.1	90.0

(a) Darco Activated Carbon, Grade KB, Manufactured by Atlas Chemical Industries dose of less than 40 mg/l. Color removal required a dose slightly in excess of 200 mg/l.

For MBAS and phenol, carbon doses of 500 mg/l and 430 mg/l respectively would have been required to reduce the concentration of these constituents to the objectives of the DRBC for Zone 5 of the Delaware River. It should be noted, however, that batch isotherm studies can be considered as "screening tests" only. They are, however, indicative of carbon capacities, and do establish a basis for subsequent continuous column studies. A verification of these tests with additional carbon studies was performed. The results are summarized in Section VI.

FORMULATION OF THE PILOT PLANT EVALUATION PROGRAM

The information developed from the bench scale studies and reported in this Section served two basic functions: (a) an approximation of the degree of wastewater treatability was established, and (b) the performance and evaluation program inherent in the operation of the pilot plant could be designed so as to obtain maximum benefit from the study.

The pilot plant studies, the results of which are cited in Section VI, were programmed to satisfy many objectives. The more important considerations are listed as follows:

1) A continuing characterization of all input wastewaters including those organic and inorganic substances which affect process operation.

2) Monitoring of the neutralization system with respect to chemical demand, buffering capacity of the combined wastewaters, and operating characteristics of the process.

3) Analyzing the primary-clarifier with regard to process efficiency as a function of various operation conditions, nature of the accumulated sludge, and quality of the primary effluent.

4) Evaluation of the mixed liquor in the aeration basin, including the response of the microbial population to varying conditions of organic and hydraulic loadings, temperature, oxygen tension levels, suspended solids concentrations, and other environmental factors.

5) Determination of the efficiency of secondary clarification at various organic loadings and hydraulic overflow rates. This includes an evaluation of the sludge settleability, the degree of thickening which is obtainable and the resulting recycle rates which are practical, and the nature and concentration of suspended materials remaining in the effluent overflow.

6) Evaluation of the nature and dewaterability of the excess sludge produced daily within the pilot plant system. This includes primary sludge consisting of settled suspended materials which were present in the raw wastewaters, chemical sludge resulting from chemical coagulation and precipitation as well as certain substances which come out of solution during changes in pH, and excess biological sludge resulting from microbial synthesis and replication.

7) Application of miscellaneous tertiary or effluent polishing processes within the treatment system and estimating their application in removing residual and conservative substances present in the secondary effluent.

8) A detailed characterization of the effluent from the unit processes at each operating condition. It is necessary to define the processes within the system in terms of efficiency, operating constraints, and general limitations. The final effluent must be similarly defined, with the range of resulting effluent quality being considered in terms of the regulatory criteria.

Operating Factors

The factors of operating variables and ranges, necessary analytical tests for each system component, operating schedules, and duration of anticipated tests as conceived at the termination of the bench scale studies are considered herein and will be discussed individually.

Operating Variables and Ranges

It was necessary to measure the response of the pilot plant system to various hydraulic and organic loadings, with the intent of translating this information into basic design criteria for the prototype plant. Based on characterization and the treatability results reported in this chapter, the following loading conditions were scheduled to be applied to the biological system.

Operating Condition 1

Organic loading = 0.2 lbs BOD_5/lb MLVSS/day
General conditions: BOD_5 = 350 mg/l
Detention time = 18 hours
MLVSS = 2300 mg/l
Flow = 18 gpm to individual aeration tank

Operating Condition 2

Organic loading = 0.5 lbs BOD_5/lb MLVSS/day
General conditions: BOD_5 = 350 mg/l
Detention time = 12 hours
MLVSS = 1400
Flow = 25 gpm to individual aeration tank

Operating Condition 3

Organic loading = 0.8 lbs BOD_5/lb MLVSS/day
General conditions: BOD_5 = 350 mg/l
Detention time = 6 hours
MLVSS = 1750 mg/l
Flow = 50 gpm to individual aeration tank

Operating Condition 4

Organic loading = 1.2 lbs BOD_5/lb MLVSS/day
General conditions: BOD_5 = 350 mg/l
Detention time = 3 hours
MLVSS = 2330
Flow= 50 gpm to individual aeration tank

These loadings were obtained either by operating the three aeration basins in parallel or in series, depending on the required flow rate and other operational considerations. The pilot plant is designed to allow parallel operations whereby each aeration basin can be subjected to the same hydraulic and/or organic load, while environmental conditions can be varied as required in the individual cells.

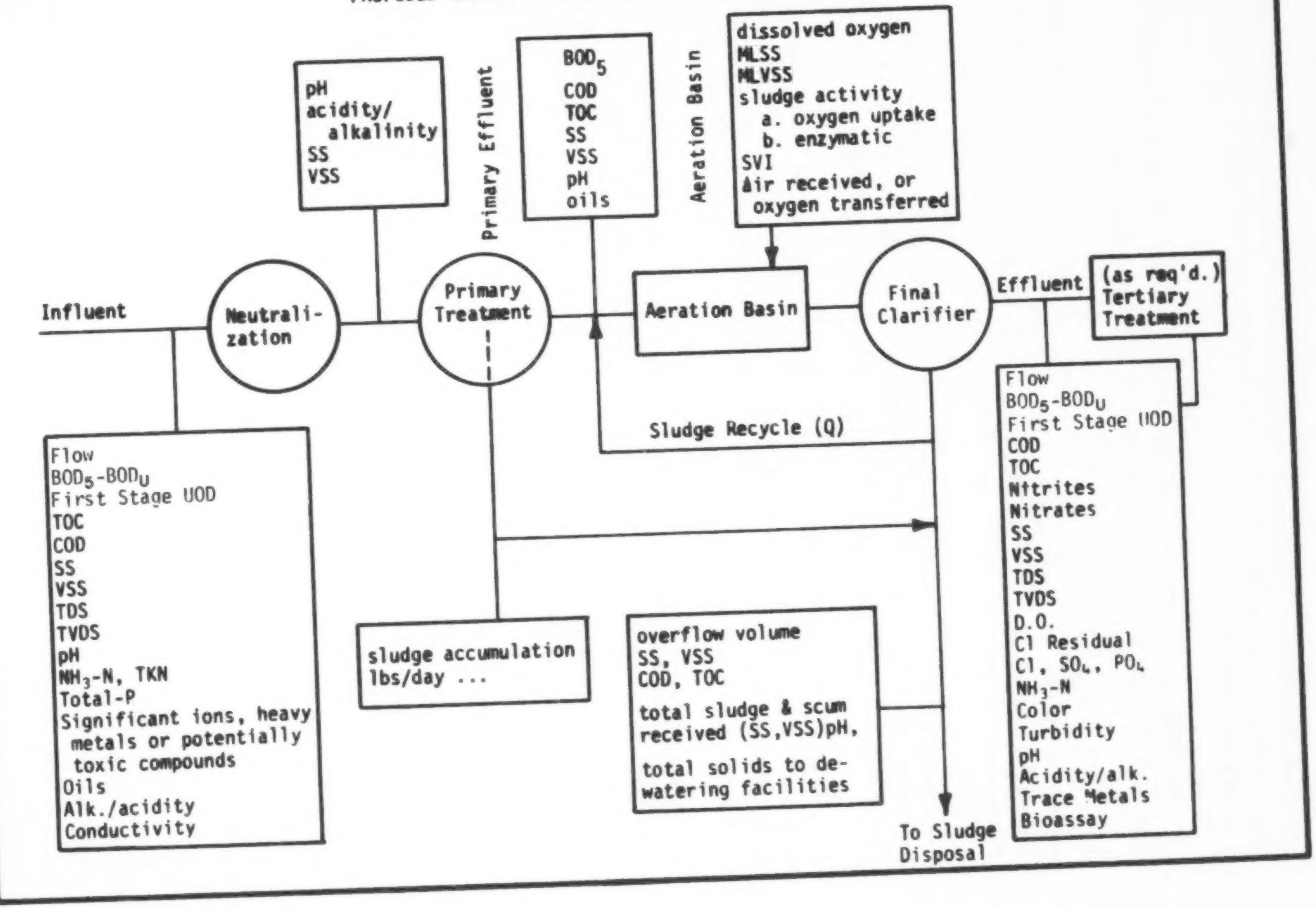
Analytical Tests

A tentative test program for the pilot plant program is shown in Figure 52. Although subsequent modifications were necessary, this tabulation provided a general testing format which included those analyses deemed necessary to properly evaluate the pilot program and to formulate the design basis for the full-scale treatment system. As indicated in this Figure, there are six major testing points within the system train, each point including those analyses necessary to evaluate the specific unit process or treatment component. These points will be discussed individually:

1.) Plant Influent - The characteristics of the raw waste were evaluated at the point where the stored industrial and municipal wastes were blended with the DuPont Chambers Works waste in the equalization basin. This characterization

Figure 52

PROPOSED CONTROL TESTS FOR PILOT PLANT EVALUATION


Influent
Neutralization
pH
acidity/alkalinity
SS
VSS
Primary Effluent
Primary Treatment
BOD5
COD
TOC
SS
VSS
pH
oils
Aeration Basin
dissolved oxygen
MLSS
MLVSS
sludge activity
a. oxygen uptake
b. enzymatic
SVI
air received, or oxygen transferred
Aeration Basin
Final Clarifier
Effluent
(as req'd.) Tertiary Treatment
Flow
BOD5-BODU
First Stage UOD
TOC
COD
SS
VSS
TDS
TVDS
pH
NH3-N, TKN
Total-P
Significant ions, heavy metals or potentially toxic compounds
Oils
Alk./acidity
Conductivity
Sludge Recycle (Q)
sludge accumulation lbs/day ...
overflow volume
SS, VSS
COD, TOC
total sludge & scum received (SS,VSS)pH,
total solids to dewatering facilities
To Sludge Disposal
Flow
BOD5-BODU
First Stage UOD
COD
TOC
Nitrites
Nitrates
SS
VSS
TDS
TVDS
D.O.
Cl Residual
Cl, SO4, PO4
NH3-N
Color
Turbidity
pH
Acidity/alk.
Trace Metals
Bioassay

included the necessary organic and inorganic analyses, solids concentrations, oils, acidity, and specific detection of substances related to potential biological toxicity and nutrient demand.

2.) Neutralization Effluent -The liquid discharged from the neutralization tanks was monitored for pH, acidity or alkalinity, and suspended solids.

3.) Reactor-Clarifier Effluent - The primary effluent was analyzed for organic substances, solids, pH, oils, and other constituents as required, recognizing that the wastewater at this point represents the actual input to the biological portion of the system.

4.) Aeration Basin - The mixed liquor in the aeration basin was analyzed to determine environmental conditions such as pH, temperature, and oxygen tension, sludge concentration, biological activity, and other tests as required.

5.) Sludge Holding Tank - The accumulated primary, chemical, and excess activated sludge was pumped to a temporary holding tank, where samples were withdrawn and characterized according to chemical constituents biological viability, and dewaterability.

6.) Secondary and/or Tertiary Effluent - The final effluent from the pilot plant was analyzed in accordance with those tests cited in Figure 52. This included all analyses necessary to evaluate the efficiency of the total system, to determine the fate of individual constituents, and to estimate the quality of the treated effluent with respect to that allowable.

Operating Schedules

The operating schedules for the changing of loading conditions or alteration of process variables depended primarily on the response of the system to a given condition, as indicated by the data. Generally, a given load or set of environmental conditions was imposed on the total system or an individual component, until a "steady state" or a "quasi-steady state" response had been obtained. This meant that the variation of system responses to a given input, i.e., process efficiency, oxygen utilization, etc., had been minimized and varied only with the nominal changes in the raw waste.

Duration of Anticipated Tests

At the outset of the pilot plant tests, a general time table was outlined for the plant operation based on the treatability studies and on past experience. Some duration from this time frame was imposed, as explained in Section VI.

It normally takes three to four weeks for a biological population in an activated sludge aeration basin to become acclimated to chemical and refinery wastewaters. Once acclimation is obtained, an additional two to three weeks is required for the system to become equilibrated to a defined loading level. One or two more weeks are then required to evaluate properly all of the desired parameters at the level imposed. If environmental changes occur, additional time is required in order to allow the biological population to adjust to such changes. Based on the aforementioned, the estimated duration of each anticipated test was programmed as shown below:

Condition Description	Time Requirement
System startup - dye studies, etc.	3.0 months
Acclimation of biological culture to wastewater	1.0 month
Operating Condition No. 1 (Lowest organic and hydraulic loading - including equilibration time) -winter and summer conditions.	1.5 months
Operating Condition No. 2 - Summer and winter conditions	2.0 months
Operating Condition No. 3 - Summer and winter conditions	2.0 months
Operating Condition No. 4 - (highest organic and hydraulic loading - including equilibration time) -summer and winter conditions.	2.5 months
General evaluation of various environmental conditions.	1.0 month
General process evaluation; auxiliary studies, operational and control studies.	6 to 12 months

As previously mentioned and as will be noted in Section VI, several alterations in process operations and testing procedures were made to fit the situation. The general format as mentioned here, however, proved applicable in most instances. Although construction of the pilot plant by Zurn Environmental Engineers occurred concurrently with the bench scale studies reported herein, the pilot facility did not come fully operational until the termination of the bench scale studies.

REFERENCES

1. Standard Methods, 12th Ed., American Public Health Association (1965)

2. Eckenfelder, W. W., and Ford, D. L., Water Pollution Control - Experimental Procedures for Process Design, Pemberton Press, Austin, (1970).

3. Wallace, A. T., "Analysis of Equalization Basins," Journal of the Sanitary Engineering Division, Proceedings of the American Society of Civil Engineers, Dec. (1968).

SECTION VI

PILOT PLANT TREATABILITY STUDIES

The development of design criteria and an economic evaluation of the various wastewater treatment processes can be effected to a limited extent using a bench scale testing approach as reported in Section V. However, in dealing with complex industrial-municipal wastewater such as that entering the Deepwater Regional Treatment System, bench scale studies are constrained because of the very nature of their operations. Hence, a pilot scale wastewater investigation program was deemed necessary to evaluate treatment processes under field conditions. Engineering-Science and Zurn Environmental Engineers designed, constructed, and operated a 50 gpm biological treatment pilot plant for the purpose of developing these design criteria. The intent of this Section is to describe the design and subsequent modifications of the pilot plant, discuss its operation and control, outline the data analysis techniques used in determining design criteria, and evaluate the wastewater treatment processes tested during the pilot plant program.

PILOT PLANT DESIGN AND MODIFICATIONS

Description of the Pilot Plant Facilities

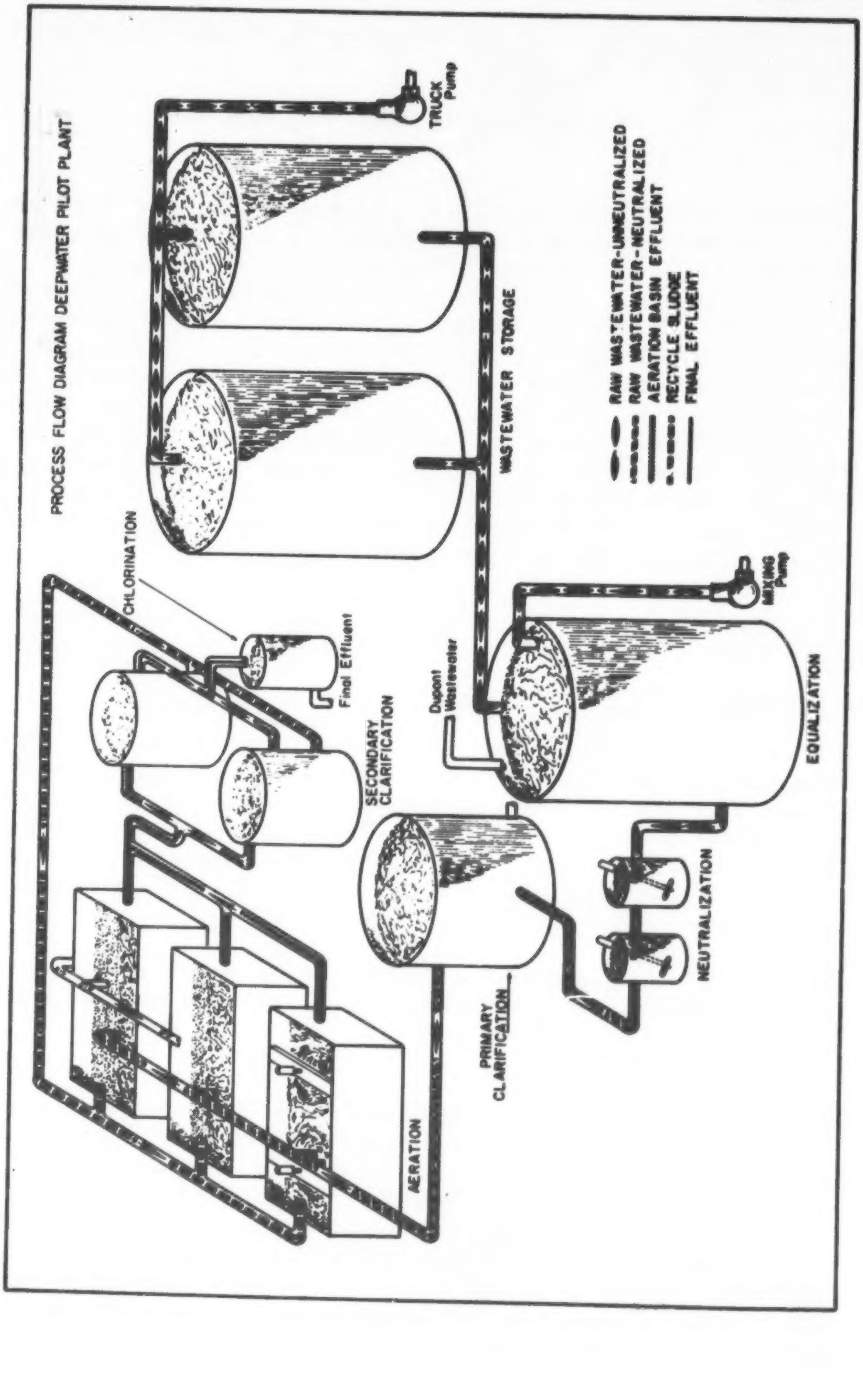
The pilot plant treatment processes include equalization, neutralization, primary clarification, aeration, secondary clarification and chlorination as shown in Figure 53. In addition, other treatment processes have been demonstrated at the pilot plant - including centrifugation, vacuum filtration, filter press dewatering, carbon adsorption, chemical treatment, aerobic sludge digestion, and effluent filtration. These ancillary tests were completed utilizing pilot scale equipment temporarily installed at the pilot plant site.

The "as built" construction drawings for the pilot plant facility are shown in Figures 54 through 59. Photographs of the pilot plant are shown in Figure 60.

Wastewater Storage

The Deepwater Pilot Plant was designed on a maximum throughput of 50 gpm. Of the 72,000 gallons of wastewater treated per day, approximately half was transported to the pilot plant via tank truck, while the remaining wastewater was pumped directly to the plant from the duPont Chambers Works outfall. Wastewater storage facilities were provided for the transported wastes utilizing two wood stave storage tanks, each with a working capacity of 82,150 gallons. Wood construction was selected because of the corrosive nature of some of the industrial wastewaters. A 250 gpm truck unloading pump was provided to off-load the 5,600 gallon tank trucks. All necessary piping included within this system was of fiberglass

Figure 53



PROCESS FLOW DIAGRAM DEEPWATER PILOT PLANT

TRUCK Pump
WASTEWATER STORAGE
CHLORINATION
Final Effluent
SECONDARY CLARIFICATION
Dupont Wastewater
MIXING Pump
EQUALIZATION
PRIMARY CLARIFICATION
NEUTRALIZATION
AERATION
RAW WASTEWATER-UNNEUTRALIZED
RAW WASTEWATER-NEUTRALIZED
AERATION BASIN EFFLUENT
RECYCLE SLUDGE
FINAL EFFLUENT

Figure 54

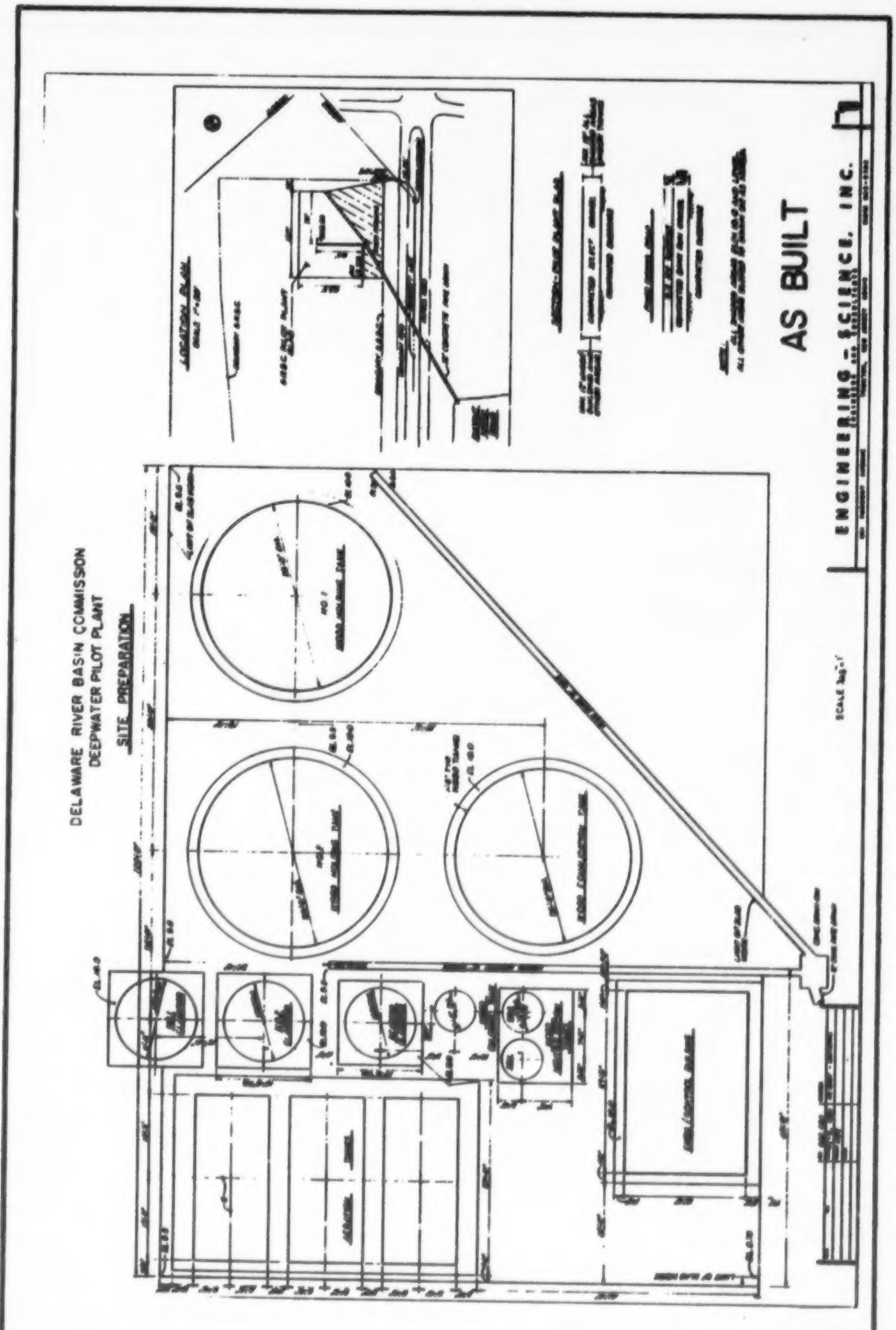

DELAWARE RIVER BASIN COMMISSION
DEEPWATER PILOT PLANT
SITE PREPARATION
LOCATION PLAN
AS BUILT
ENGINEERING - SCIENCE, INC.

Figure 55

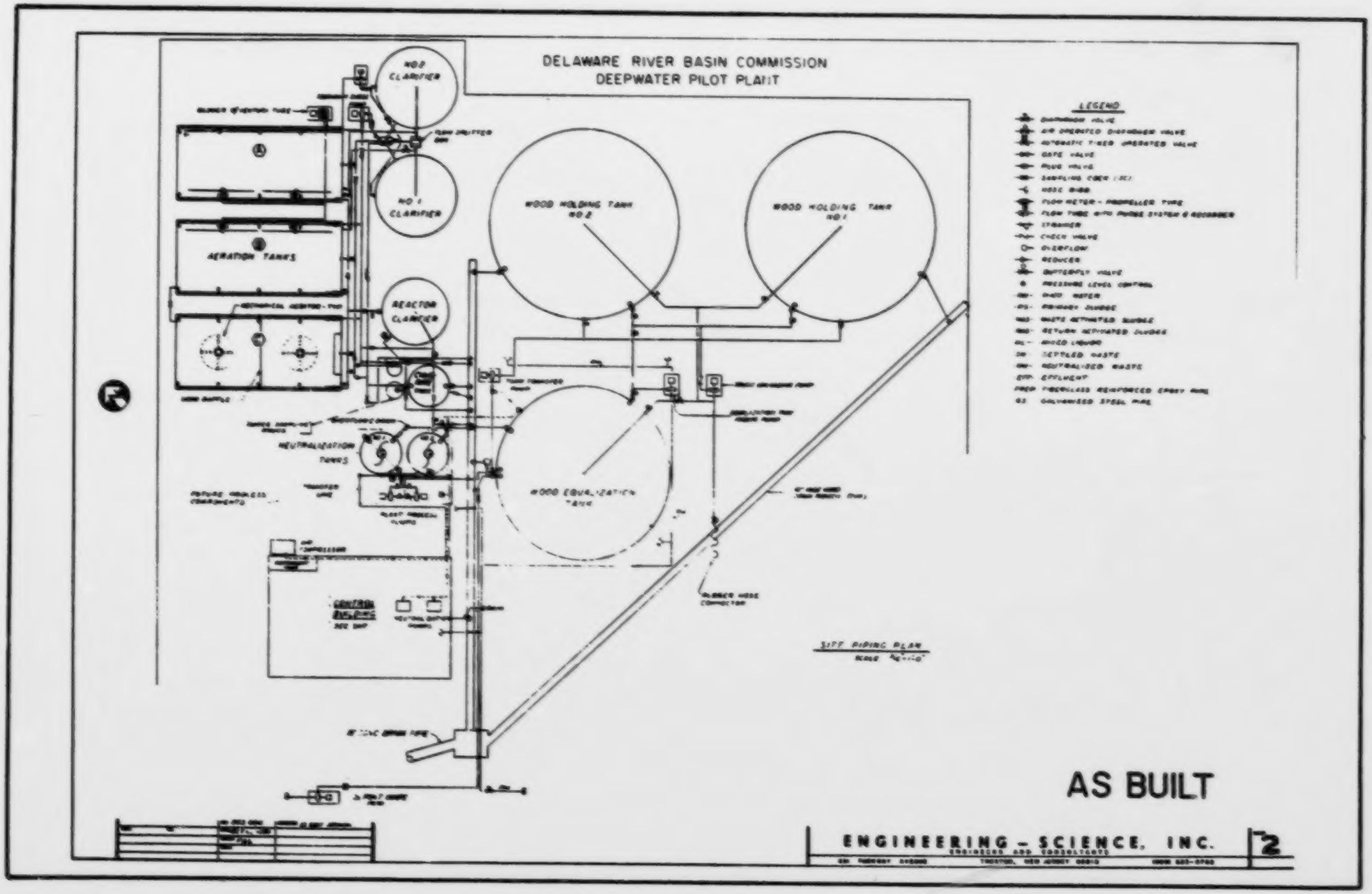
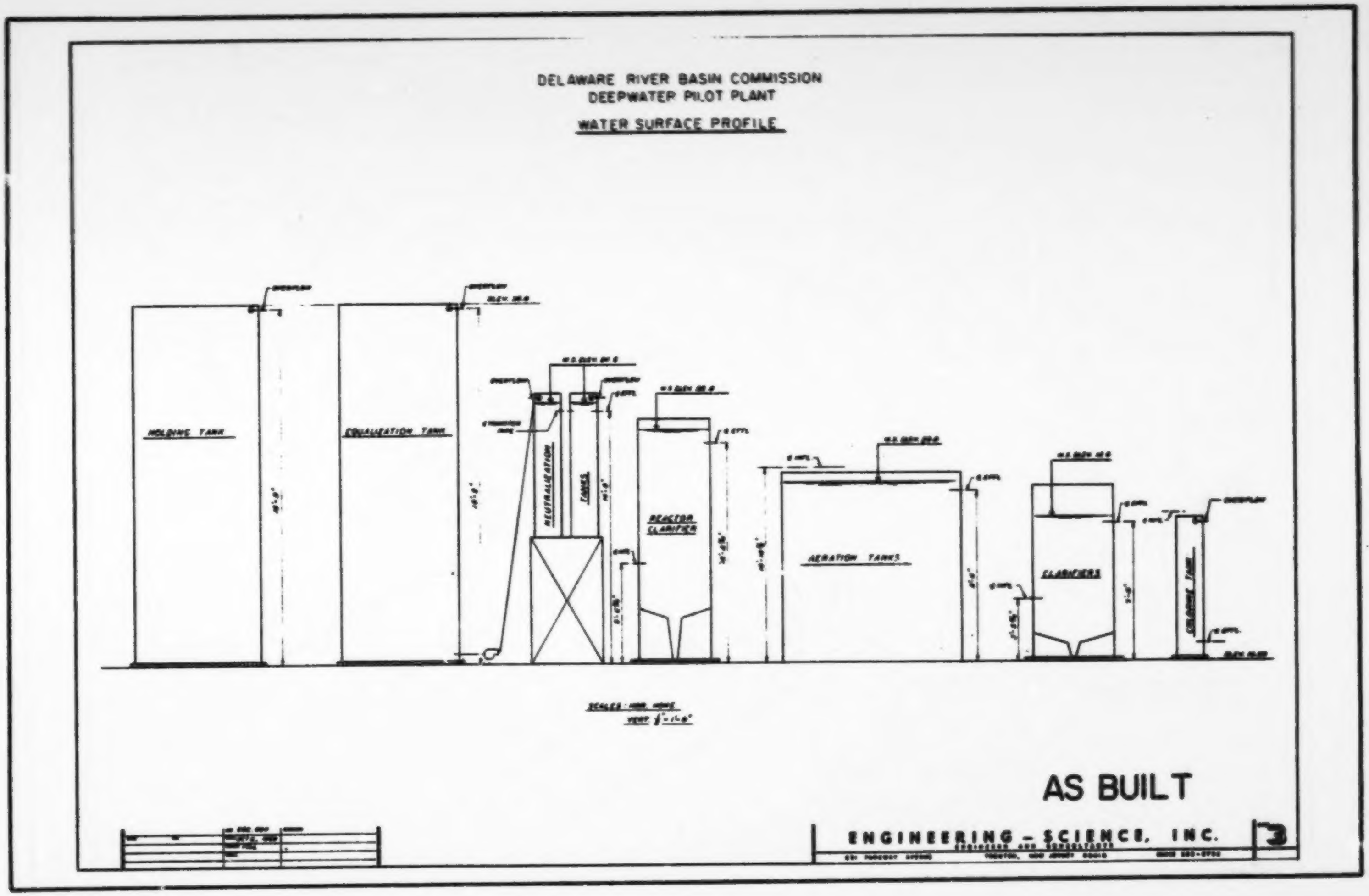

DELAWARE RIVER BASIN COMMISSION
DEEPWATER PILOT PLANT
LEGEND
DIAPHRAGM VALVE
AIR OPERATED DIAPHRAGM VALVE
AUTOMATIC TIMER OPERATED VALVE
GATE VALVE
PLUG VALVE
SAMPLING COCK (SC)
HOSE BIBB
FLOW METER - PROPELLER TYPE
FLOW TUBE WITH PURGE SYSTEM & RECORDER
STRAINER
CHECK VALVE
OVERFLOW
REDUCER
BUTTERFLY VALVE
PRESSURE LEVEL CONTROL
RAW WATER
PRIMARY SLUDGE
WASTE ACTIVATED SLUDGE
RETURN ACTIVATED SLUDGE
MIXED LIQUOR
SETTLED WASTE
NEUTRALIZED WASTE
EFFLUENT
FIBERGLASS REINFORCED EPOXY PIPE
GALVANIZED STEEL PIPE
NO 2 CLARIFIER
NO 1 CLARIFIER
AERATION TANKS
REACTOR CLARIFIER
WOOD HOLDING TANK NO 2
WOOD HOLDING TANK NO 1
WOOD EQUALIZATION TANK
NEUTRALIZATION TANKS
FUTURE PROCESS COMPONENTS
CONTROL BUILDING
RUBBER HOSE CONNECTOR
SITE PIPING PLAN
AS BUILT
ENGINEERING - SCIENCE, INC.
ENGINEERS AND CONSULTANTS
TRENTON, NEW JERSEY
2

Figure 56



DELAWARE RIVER BASIN COMMISSION
DEEPWATER PILOT PLANT
WATER SURFACE PROFILE

HOLDING TANK
EQUALIZATION TANK
OVERFLOW
NEUTRALIZATION
TANKS
REACTOR CLARIFIER
AERATION TANKS
CLARIFIERS
CHLORINE TANK
SCALES: HOR. NONE
VERT. ⅛" = 1'-0"

AS BUILT

ENGINEERING - SCIENCE, INC.
ENGINEERS AND CONSULTANTS

Figure 57

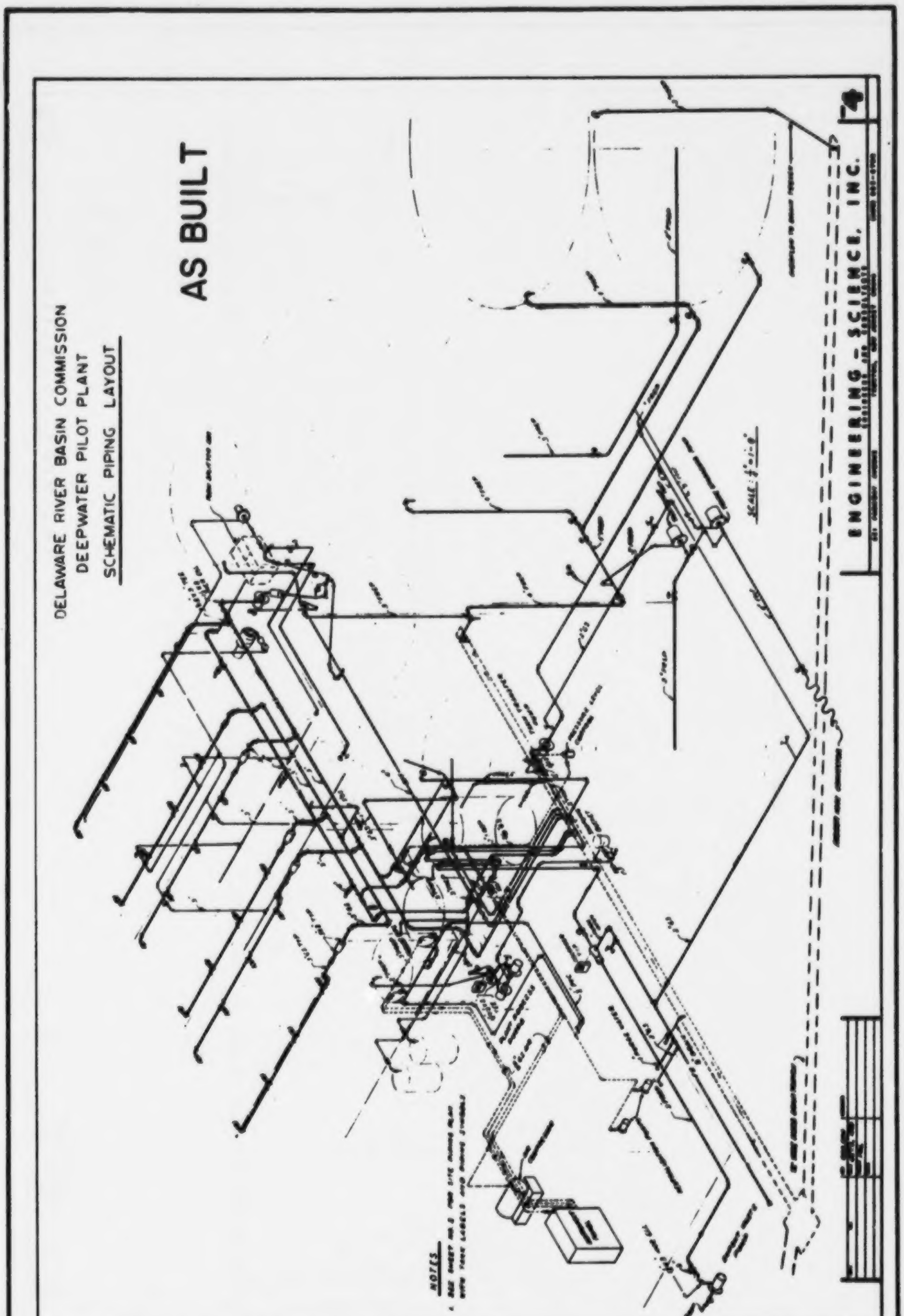

DELAWARE RIVER BASIN COMMISSION
DEEPWATER PILOT PLANT
SCHEMATIC PIPING LAYOUT
AS BUILT
NOTES
ENGINEERING - SCIENCE, INC.

Figure 58

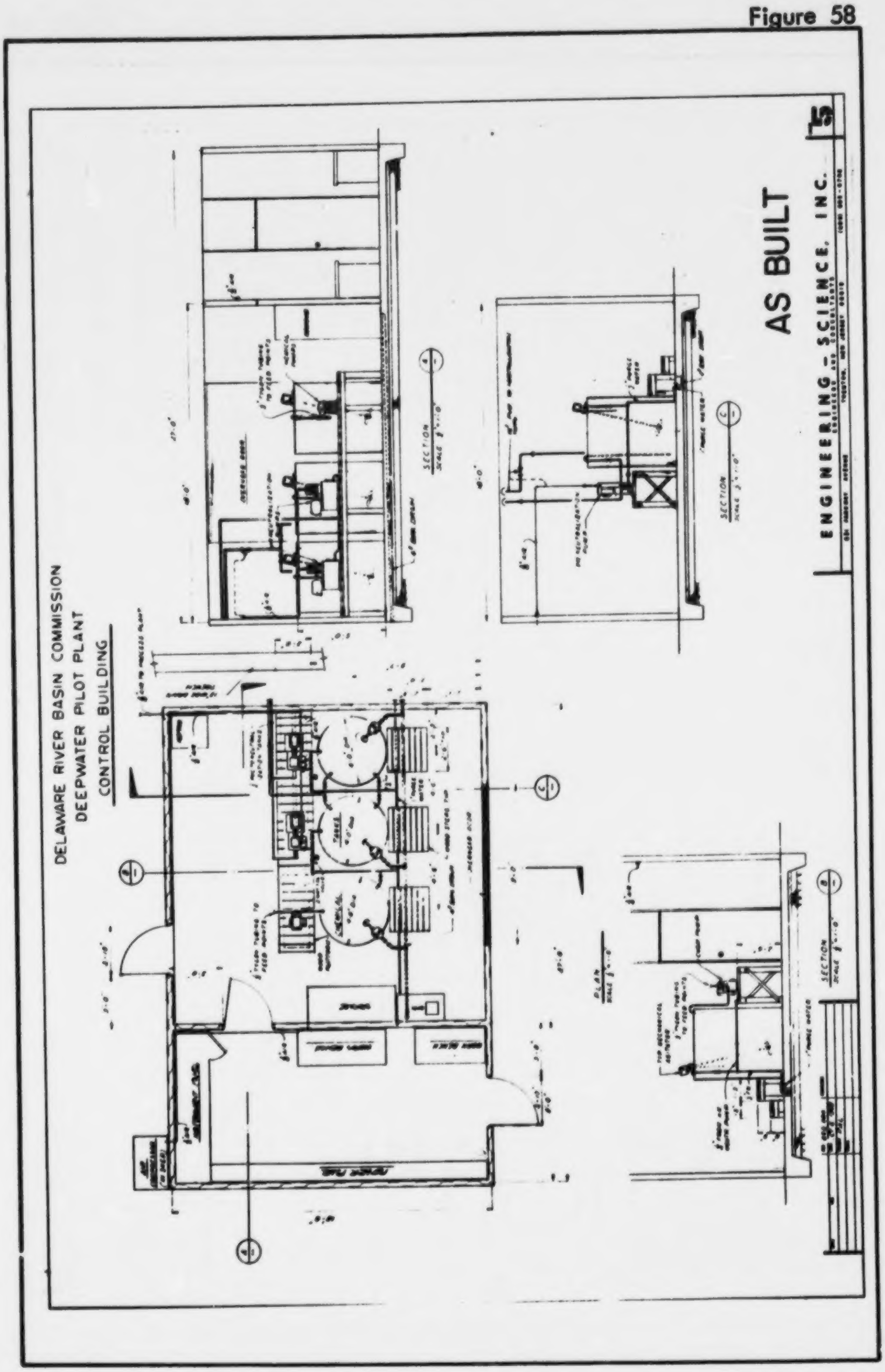
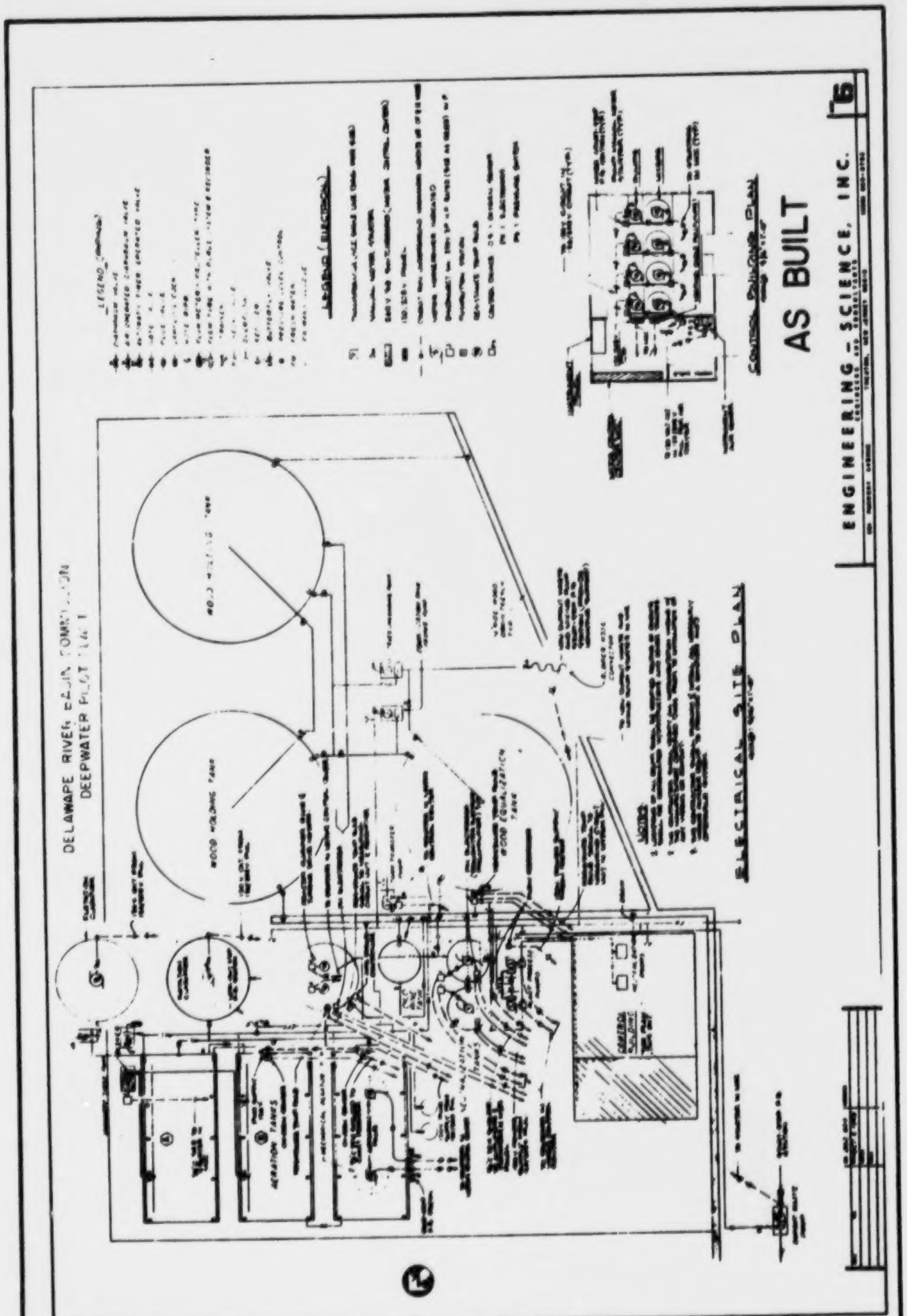
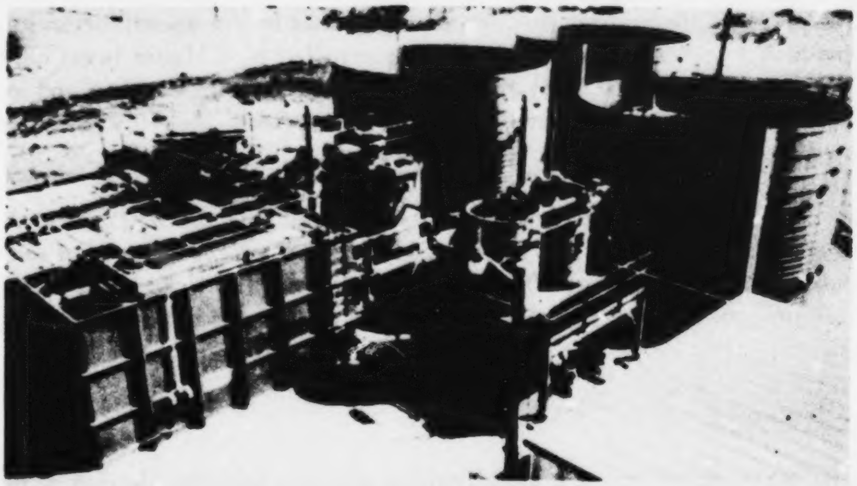
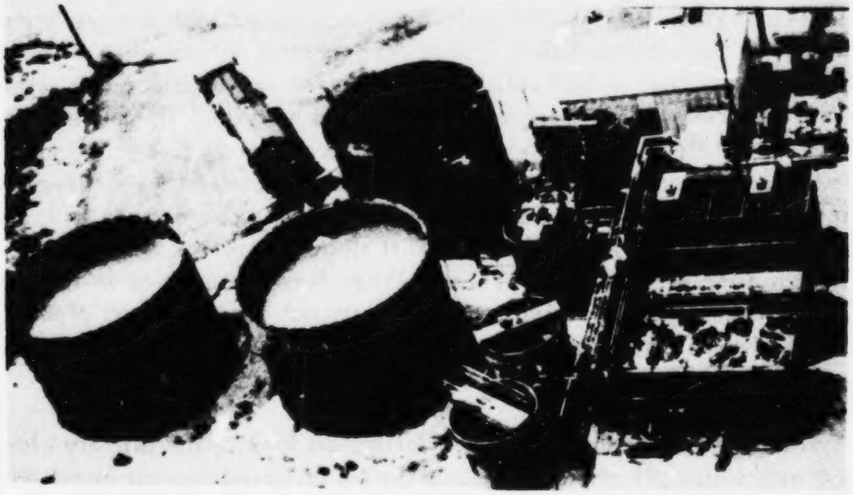

DELAWARE RIVER BASIN COMMISSION
DEEPWATER PILOT PLANT
CONTROL BUILDING
PLAN
SECTION
SECTION
SECTION
AS BUILT
ENGINEERING - SCIENCE, INC.
15

Figure 59


DELAWARE RIVER BASIN COMMISSION
DEEPWATER PILOT PLANT
ELECTRICAL SITE PLAN
CONTROL BUILDING PLAN
LEGEND (ELECTRICAL)
AERATION TANKS
EQUALIZATION TANK
AS BUILT
ENGINEERING - SCIENCE, INC.
16

PHOTOGRAPHS OF THE DEEPWATER PILOT PLANT





construction, again because of the corrosive nature of some of the wastewaters.

Equalization

The equalization tank provided a blending facility for the transported wastewater and the waste stream pumped to the pilot plant from the duPont Chambers Works plant. This tank was also of wood stave construction with a working capacity of 71,000 gallons. A minimum equalization time of 23 hours was achieved when the tank was completely full and operating at the maximum flow rate of 50 gpm. The stored wastewater was transferred from the storage tanks to the equalization tank via a 30 gpm tank transfer pump with the flow controlled by a liquid level control system on the equalization tank. By the use of this level control system and manual flow control at the point of the Chambers Works waste stream pick-up, the correct proportions of the two wastewaters could be obtained.

The original design of the equalization facility provided that the 250 gpm tank unloading pump also be used for mixing the equalization tank. Because of necessary trucking schedules, an additional 250 gpm pump was installed in parallel to the original truck unloading pump in order to allow for continuous mixing of the equalization tank.

Neutralization

The composite wastewater from the equalization tank was pumped via one of two 20 to 70 gpm process pumps, through a manual flow control and recording system and into the first stage of a two-stage neutralization system. Each neutralization tank had a working capacity of 1,200 gallons and was mixed with a three horsepower agitator. At the maximum flow rate of 50 gpm, the detention time for each stage was 24 minutes. The neutralization agent applied was high calcium slaked lime. Control for the neutralization system was implemented by a dual Honeywell pH controller-recorder with Universal Interloc pH probes and amplifiers. A loop lime slurry feed system was installed and consisted of two air operated feed valves, two positive displacement 250 psi feed pumps, and lime slurry storage tanks with agitators. Back pressure was obtained by installing the first stage reverse-acting air operated feed valve on the lime recycle effluent line. When lime was needed in the first stage tank, this valve closed, forcing the lime slurry out the open feed pipe. The second stage feed valve was installed at the point of entry on the neutralization tank and required no additional back pressure for operation.

The piping system from the second stage neutralization tank to the primary clarifier was equipped with a low pH emergency dump system utilizing two air operated valves. This system was actuated by a low pH signal from the second stage pH probe and was preset to actuate if the pH dropped below 6.0. This prevented slugs of low pH waste from entering the biological system if lime feed problems developed. An additional pH monitoring system was installed in the effluent stream of the primary clarifier as a final pH check before the waste entered the biological system.

Primary Clarification

Primary clarification was provided with a dual purpose Eimco-type reactor clarifier. This unit acted not only as a conventional clarifier but also could be used for chemical addition and flocculation as the equipment contained a central mixing turbine and flocculation well. In addition, sludge thickening space was provided at the bottom of the structure from which sludge was piped directly to the electrically operated sludge blowdown valve. Sampling ports were provided above the bottom of the vessel to allow visual determinations of the sludge blanket height. The necessary frequency for sludge blowdown could thus be determined.

The clarifier was originally sized for an overflow rate of 1,120 gpd/ft^2 at 50 gpm. However, the geometry of the center reaction well and the effluent weir assembly was such that the theoretical overflow rate "as built" was 1,529 gpd/ft^2 at 50 gpm. The working volume of the clarifier was 6,150 gallons which allowed a minimum detention time of two hours at 50 gpm.

Aeration

General - The effluent from the primary clarifier entered a manifold piping system for the wastewater distribution to the aeration tanks. Three 18,000 gallon aeration tanks were provided and piped so that they could be operated in parallel, in series, or independently as required. Each tank was equipped with two header systems, one for the influent wastewater from the primary clarifier and one for the return activated sludge from the final clarifiers. Each was configured with four feed valves spaced equidistantly along the tank, allowing wastewater and return sludge to enter the tankage at any desired point. Two of the aeration tanks were supplied with diffused aeration equipment and the remaining tank was supplied with mechanical aerators.

Mechanical Aeration Tank - The mechanical aeration tank was equipped with two Eimco-Simcar 1.5 horsepower surface type aerators. Each aerator had a surface turbine for aeration and a submerged mixing turbine five feet below the operating liquid level. The aerators were mounted on adjustable platforms in order that the submergency of the surface turbines could be varied a maximum of three inches. This aeration tank was equipped with an effluent weir box to maintain a constant liquid level at flow rates up to 100 gpm. The tank was also fitted with the necessary structural members for the placement of a wood baffle segmenting the tank in half for a 9,000 gallon aeration chamber. At a 50 gpm wastewater throughput, a minimum aeration detention time of three hours could be obtained with the baffle installed.

Diffused Aeration - The diffused aeration tanks were equipped with two air header systems per tank. Each air header, located one foot from the bottom of the tank, had eight Eimco non-clog diffuser plates. An attached Suiterbilt Rotary Blower with an adjustable speed clutch assembly provided compressed air at the rate of 200 to 350 cfm at five psi. A Fisher and Porter Flow Tube was provided to measure the air flow rate from the blower.

The liquid level for both diffused aeration tanks was initially controlled by the four inch outlet pipes located one foot from the top of each aeration tank. This outlet piping conveyed the waste to a flow splitter box and then into both final clarifiers. Subsequent modifications were made to the diffused aeration tanks so that each tank could be operated independently, utilizing the two final clarifiers separately. Under this mode of operation, the liquid level was controlled by the height of the overflow weirs in the final clarifiers.

Secondary Clarification

The effluent from the aeration tanks - as previously mentioned - was conveyed to a flow splitter box and then into the two final clarifiers. The final clarifiers were Eimco-type flotation clarifiers and were modified to serve as conventional-type clarifiers for the biological treatment studies. Each clarifier had a working volume of 5,000 gallons with a detention time of 3.3 hours and an overflow rate of 380 gpd/ft^2 at a 25 gpm throughput. A single sludge return system was initially provided to serve both clarifiers.

Modifications were made to the clarifiers to provide each of the diffused aeration tanks with a clarifier and an independent sludge return system. This configuration allowed testing of two biological systems independently and provided greater flexibility of pilot plant operations.

Chlorination

The effluent from the final clarifiers was piped to a 1,200 gallon chlorine contact tank. At the maximum flow of 50 gpm, the detention time for this system was 24 minutes. The overflow from this tank was piped to the wood box drainage system for the pilot plant.

Hydraulic Studies of the Pilot Plant Units

Dye studies were conducted at the Deepwater Pilot Plant to determine the flow characteristics of the individual units. The purpose of the studies was to insure that the pilot plant data was evaluated under known hydraulic conditions. Also, such studies permitted undesirable conditions to be detected and corrected during the initial phases of the investigation.

Procedure

The basic procedure consisted of adding a measured amount of fluorescent dye to the influent of the particular unit being evaluated and measuring the concentration of dye in the effluent as a function of time. As discussed below, the flow characteristics in the unit can be ascertained from the shape of the dye recovery versus time curve.

Two dyes were used successfully during the course of the studies. During the initial studies, pontacyl brillant pink B was utilized but unfortunately additional supplies could not be obtained and the final studies were conducted with Rhodamine B-WT. An attempt was made to use straight Rhodamine B in the aeration tank studies, but apparently bacterial decay resulted in extremely low dye recoveries and the use of that dye was discontinued.

The effluent concentration of dye was measured with a Turner Fluorometer Model Number 111 equipped with a 546 primary filter and a 590 secondary filter. The fluorometer was calibrated using serial dilutions of the respective dyes at 20°C. All samples were brought up to the calibration temperature before determining the dye concentration.

All of the dye studies were conducted while the pilot plant was treating only the Chambers Works wastewater. Because fluorescent materials are manufactured at the plant, there was a slight background concentration of fluorescence that had to be accounted for in analyzing the data.

Data Analysis

Theoretical Analysis - The main purpose of the flow studies was to determine the relative amounts of complete mixing, plug flow, and dead space that was occurring in each unit process and to compare the actual results with the desirable characteristics.

Complex mathematical models have been derived for describing various combinations of flow regimes that occur in a theoretical hydraulic system. Applications of the theoretical models to real systems have, in some cases, been quite satisfactory. The disadvantage of using complex models, however, can be attributed to the fact that the original purpose of the flow study can be lost in the complexity and accuracy of the analysis.

The method utilized in this study is based on the flow models proposed in Reference I. The basic models can be presented graphically as shown in Figures 61 through 63.

Figure 61 shows the effect of dead space on a completely mixed flow system. It can be shown theoretically that approximately 63 percent of the dye added to a completely mixed system with no dead space will be recovered after one detention time:

Figure 61

THEORETICAL DYE RECOVERY CURVES FOR A COMPLETELY MIXED SYSTEM WITH VARYING AMOUNTS OF DEAD SPACE

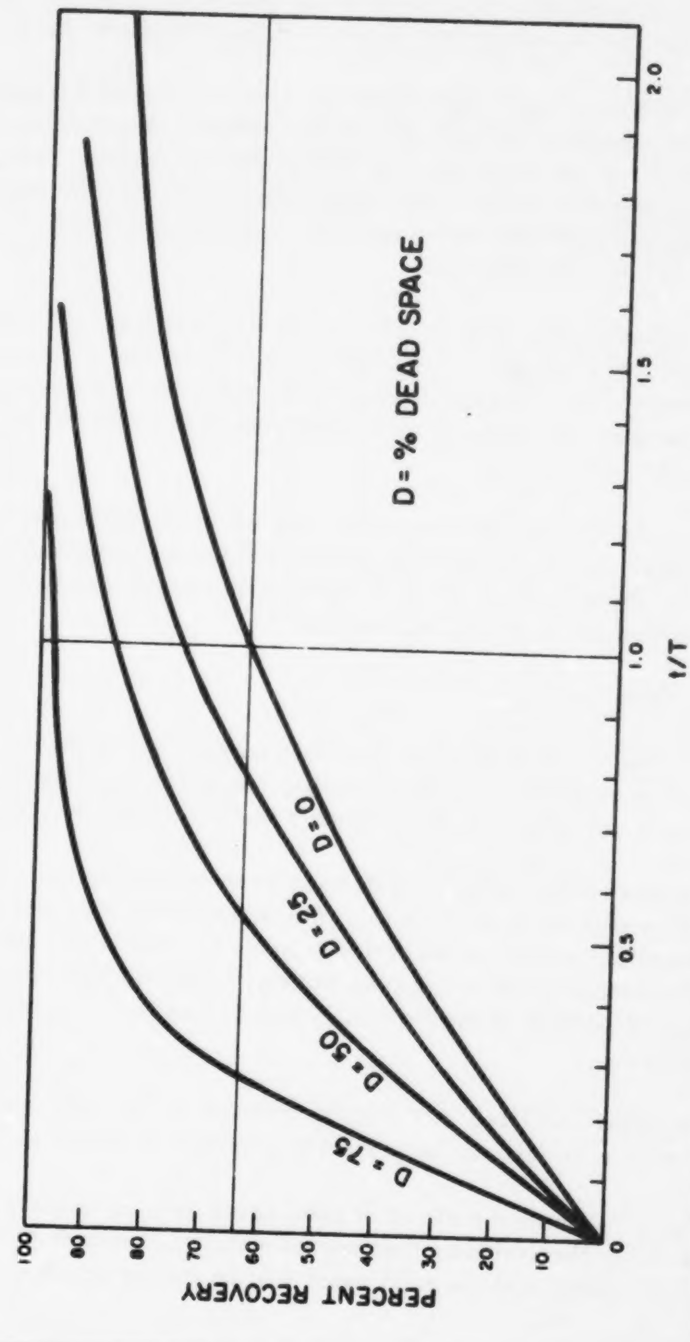
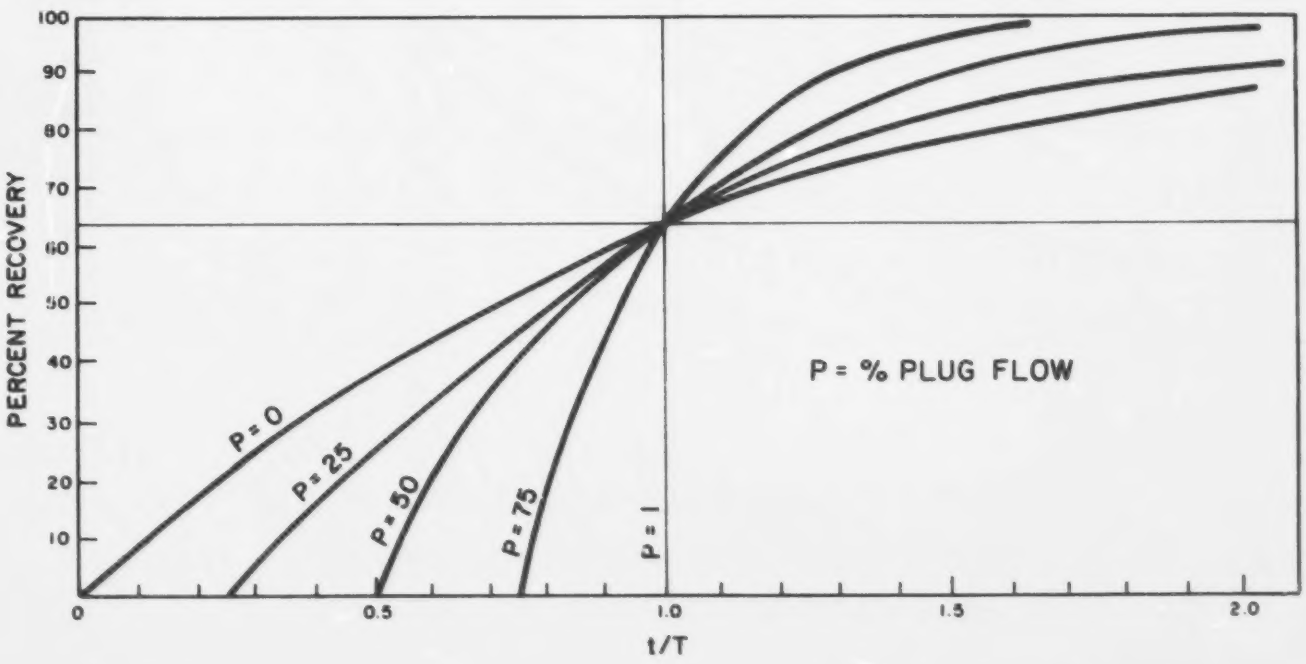
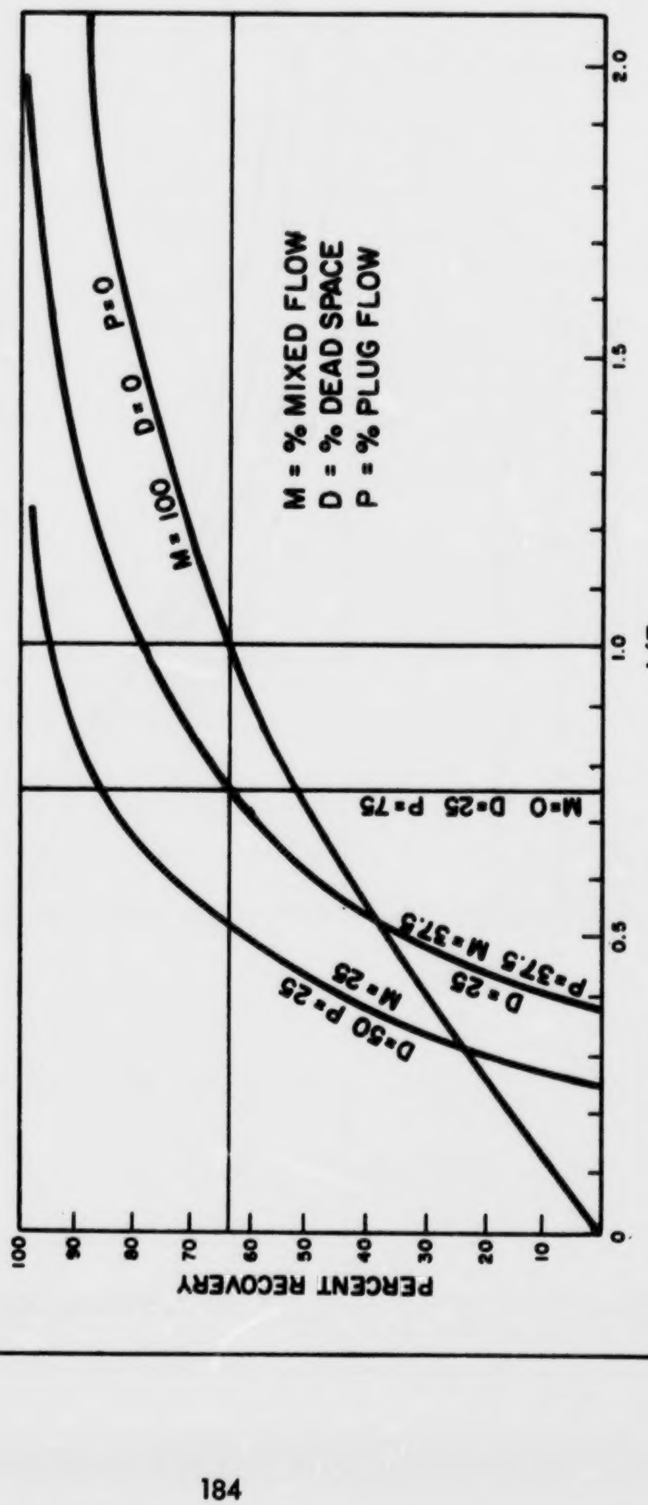


Figure 62

THEORETICAL DYE RECOVERY CURVES FOR
A COMPLETELY MIXED SYSTEM WITH VARYING
AMOUNTS OF PLUG FLOW



THEORETICAL DYE RECOVERY CURVES FOR A COMPLETELY MIXED SYSTEM WITH VARYING AMOUNTS OF DEAD SPACE AND PLUG FLOW



$$\text{Percent Recovery} = 100\,(1 - e^{-t/T}) = 100\,(1 - e^{-1.0}) = 63 \qquad \text{(VI-1)}$$

where:

T = detention time
t = actual measured time interval

To determine the amount of dead space in a completely mixed system, it is necessary only to determine what fraction of a detention time 63 percent of the dye is recovered. The remaining fraction is then equal to the fraction of dead space in the vessel. As shown in Figure 61, if 63 percent of the dye is recovered at t/T = 0.75, the amount of dead space is 0.25 or 25 percent.

Figure 62 shows the effect of plug flow on a completely mixed system. In this case, all of the curves pass through 63 percent dye recovery at t/T = 1.0, but the curves originate at various points on the abscissa. The fraction of plug flow is equal to the starting point on the abscissa. For example, a completely mixed system having 50 percent plug flow would have a recovery curve originating at t/T = 0.50, as shown in Figure 62.

Figure 63 shows various combinations of plug flow and dead space in a completely mixed system. The determination of the relative amounts of the three characteristics proceeds exactly as for the individual cases. For example, a system having 25 percent dead space would show 63 percent dye recovery at t/T = 0.75. If, in the same system, the remaining volume - i.e., the effective volume - were divided evenly between completely mixed and plug flow, the curve would originate at t/T = 0.375 - i.e., one half of (1.0 - 0.25).

Interpretation of Field Data - As discussed in the Procedure Section, the concentration of dye in the effluent from a particular unit was measured versus time. This was then plotted with concentration as the ordinate and time as the abscissa. For ease of analysis, both parameters are "normalized" by dividing the concentration C by C_o and the time t by T where:

C and t = actual concentration of dye after a particular time interval t

$$C_o = \frac{\text{weight of dye added}}{\text{theoretical volume of tank}}$$

$$T = 1.0 \text{ detention time} = \frac{\text{volume}}{\text{flow}}$$

The actual percent recovery of dye is then equal to the area under the curve of C/C_o versus t/T. A percent recovery versus time curve can be established by integrating the curve for various time intervals. The latter curve is constructed by assuming that the area under the concentration versus time curve is equal to 100 percent dye recovery rather than the actual dye recovery.

Results

Dye recovery curves for each of the unit processes investigated are presented in Figures 64 through 76. Two different curves are presented in each figure: the left ordinate refers to the normalized concentration C/C_o, and the right ordinate refers to the percent recovery assuming 100 percent dye recovery at $t/T = \infty$. For both curves, the abscissa is the normalized time interval as t/T.

Moreover, each figure contains the resulting flow characteristics in terms of the relative percent of dead space, plug flow, and completely mixed flow. The resulting flow characteristics are also summarized in Table 25.

Summary

Equalization - The results for the equalization tank indicate tnat adequate mixing and circulation are achieved by using a high capacity recycle pump. The data showed that only 23 percent of the tank was unused or dead space and the remaining volume had completely mixed characteristics.

Neutralization - The results for both one and two stage neutralization confirmed that both tanks had completely mixed flow characteristics.

Reactor-Clarifier- The data for the reactor-clarifier were practically identical with and without the reaction turbine in service. The results indicated that the tank was approximately 85 percent mixed with 12 percent plug flow and three percent dead space. Although it would be desirable to have a higher fraction of plug flow, clarifiers typically perform satisfactorily with 10 to 20 percent plug characteristics.

Normally, a clarifier would have a higher fraction of dead space than indicated by these results. However, because the entire center section of the vessel is designed as a completely mixed reactor, the small amount of dead space would be expected. The limited amount of dead space is quite significant because it indicates that there is relatively little space available for conventional sludge storage and thickening.

Aeration Tanks - The results for the aeration tanks were essentially identical for diffused air and mechanical aeration equipment. Both systems had 100 percent completely mixed characteristics.

Secondary Clarification - The first dye study was conducted on a single clarifier with an overflow of 25 gpm and an underflow or sludge recycle rate of 25 gpm for a total flow of 50 gpm to the tank. Ideally, under such conditions 50 percent of the dye would be recovered in each stream. Although the data indicated that the recoveries were 38 and 30 percent respectively for the overflow and underflow, the most significant result was the complete lack of plug flow for

Figure 64

DYE STUDY FOR EQUALIZATION TANK

21 MARCH 1970

C_0 = 0.246 MG/L Q = 45 GPM T = 1435 MIN.

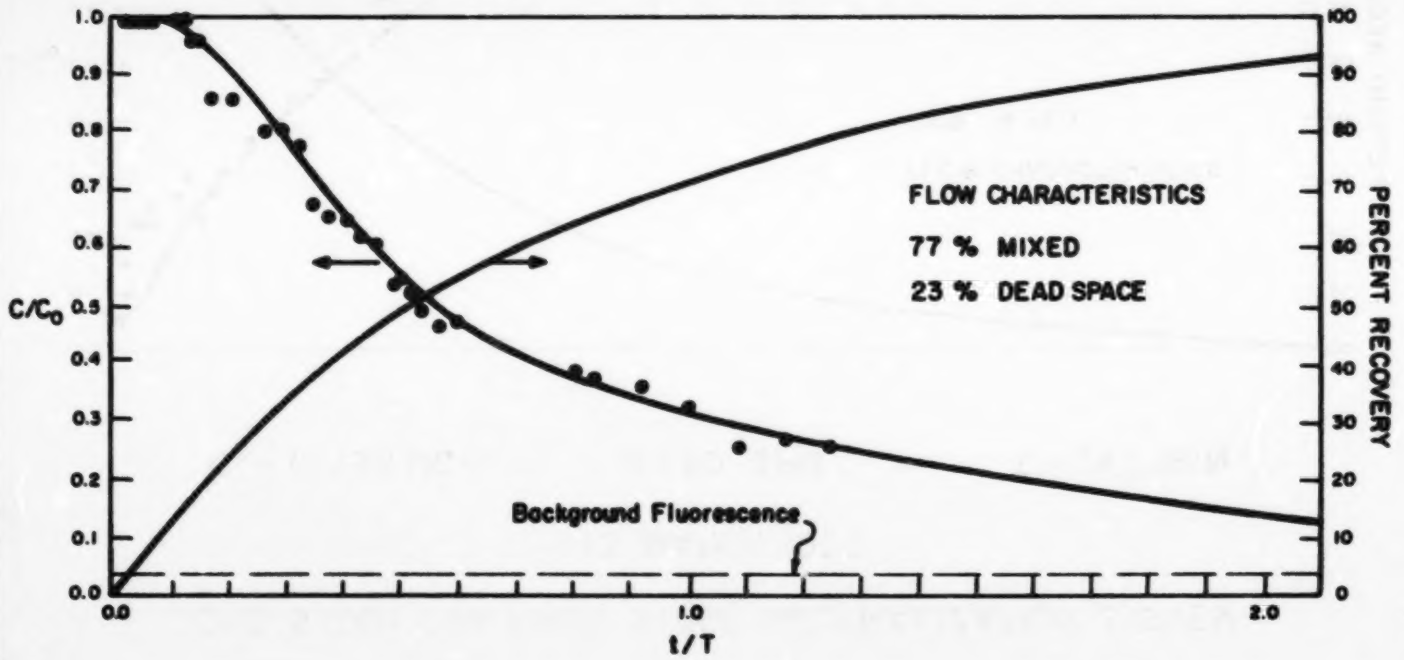


Figure 65

DYE STUDY FOR FIRST STAGE NEUTRALIZATION SYSTEM

13 MARCH 1970

C_0 = 0.536 MG/L   Q = 50 GPM   T = 24.7 MIN.

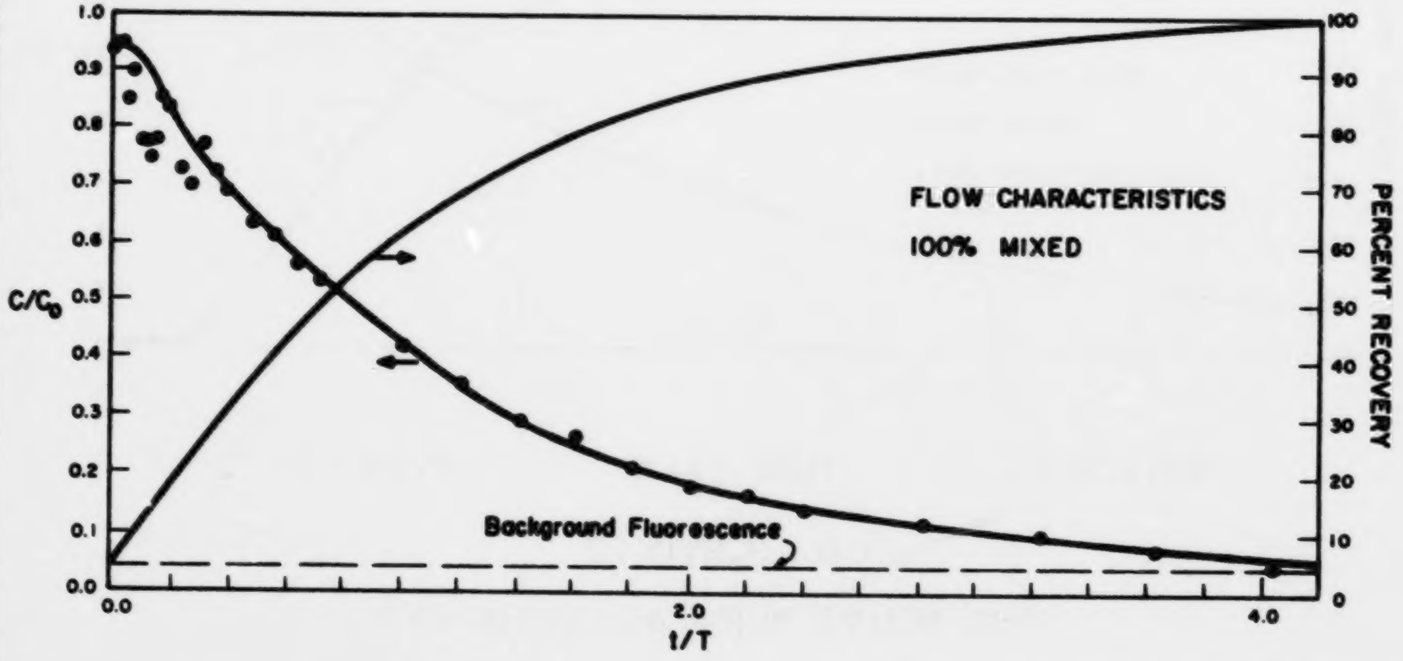


Figure 66

DYE STUDY FOR TWO-STAGE NEUTRALIZATION SYSTEM

13 MARCH 1970

$C_0 = 0.273$ MG/L   Q = 50 GPM   T = 48.5 MIN.

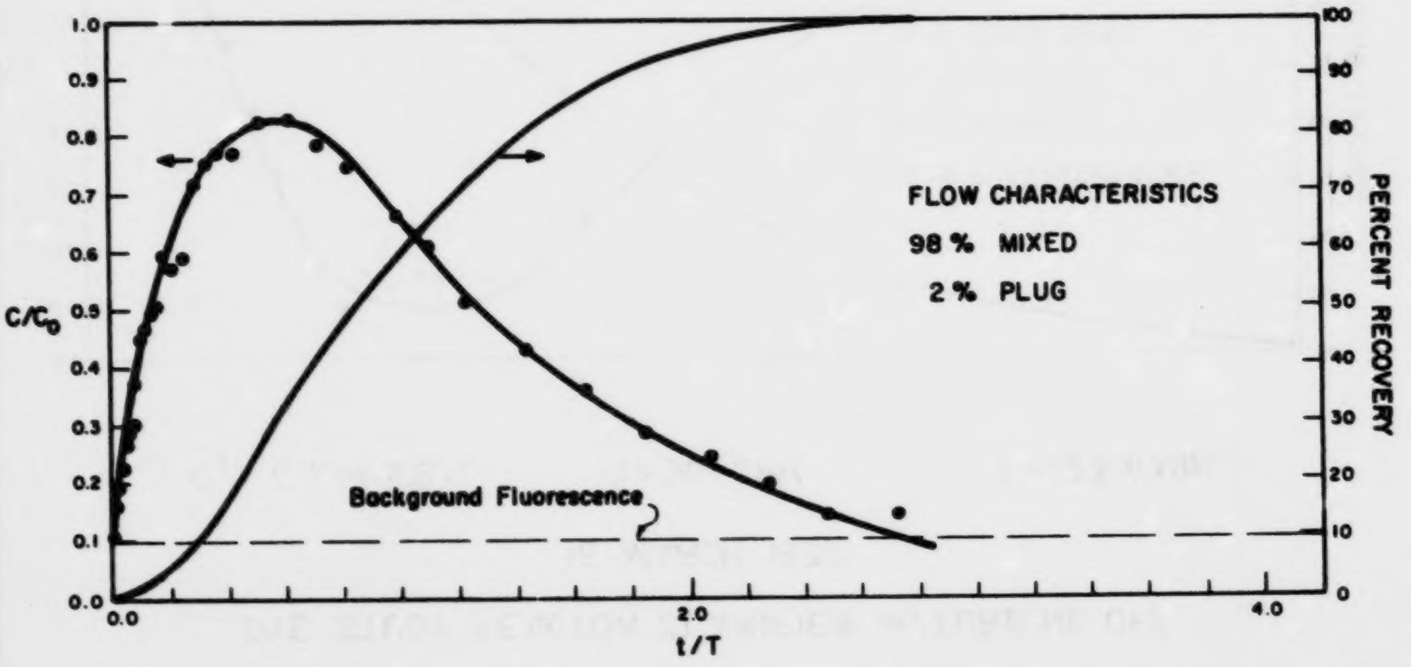
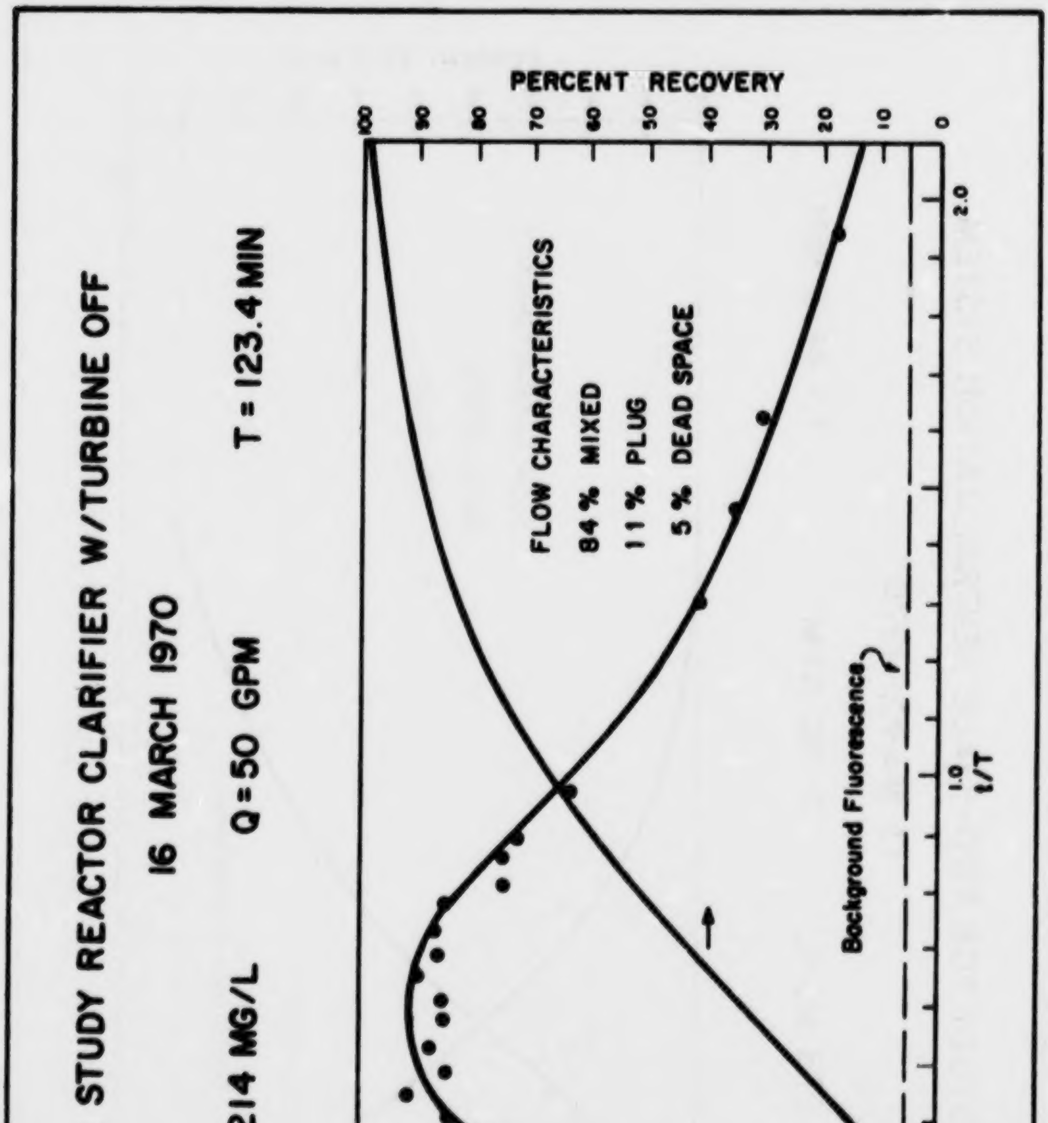


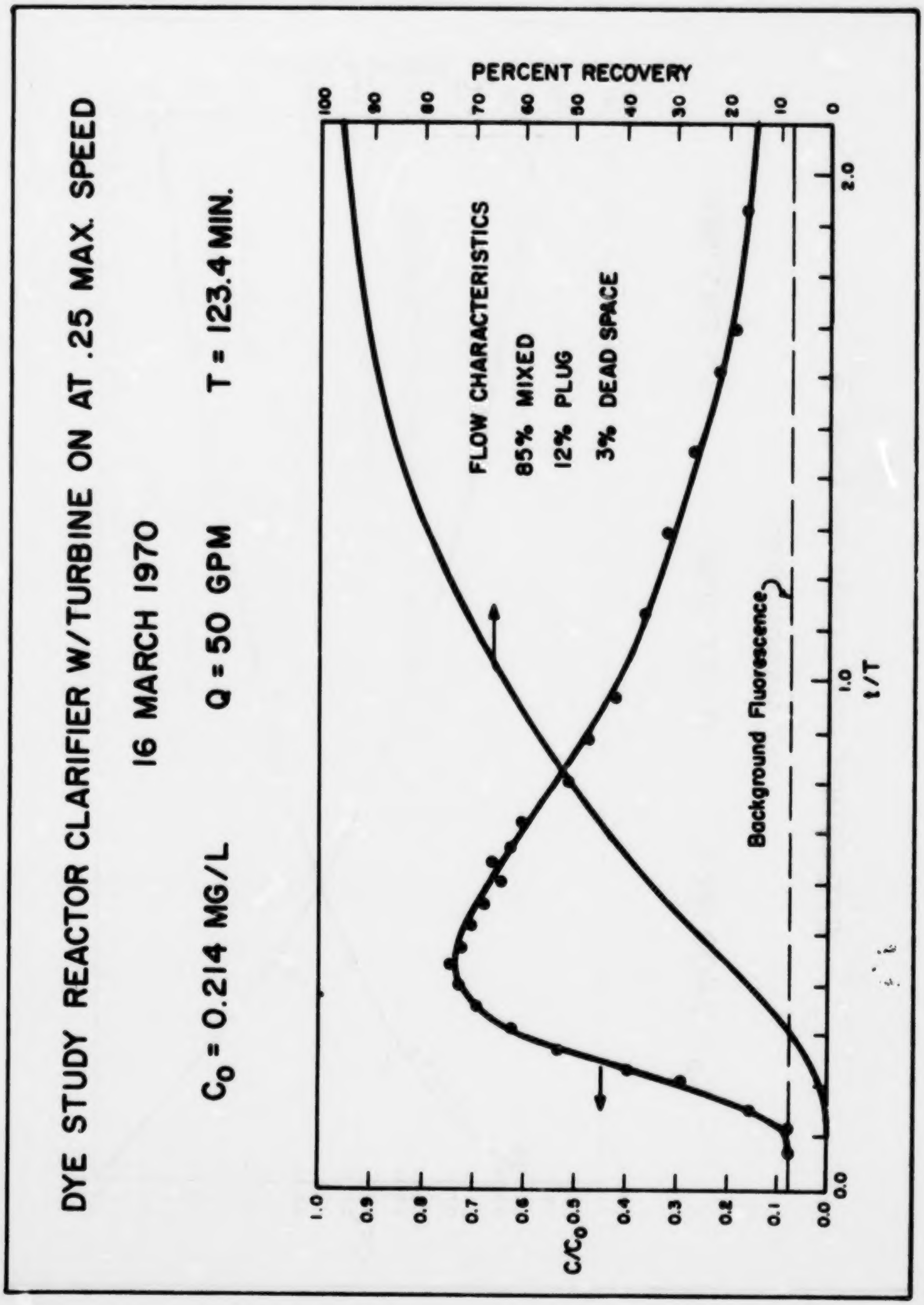
Figure 67


DYE STUDY REACTOR CLARIFIER W/TURBINE OFF

16 MARCH 1970

C_0 = 0.214 MG/L Q = 50 GPM T = 123.4 MIN

Figure 68



DYE STUDY REACTOR CLARIFIER W/TURBINE ON AT .25 MAX. SPEED

16 MARCH 1970

C_0 = 0.214 MG/L Q = 50 GPM T = 123.4 MIN.

Figure 69

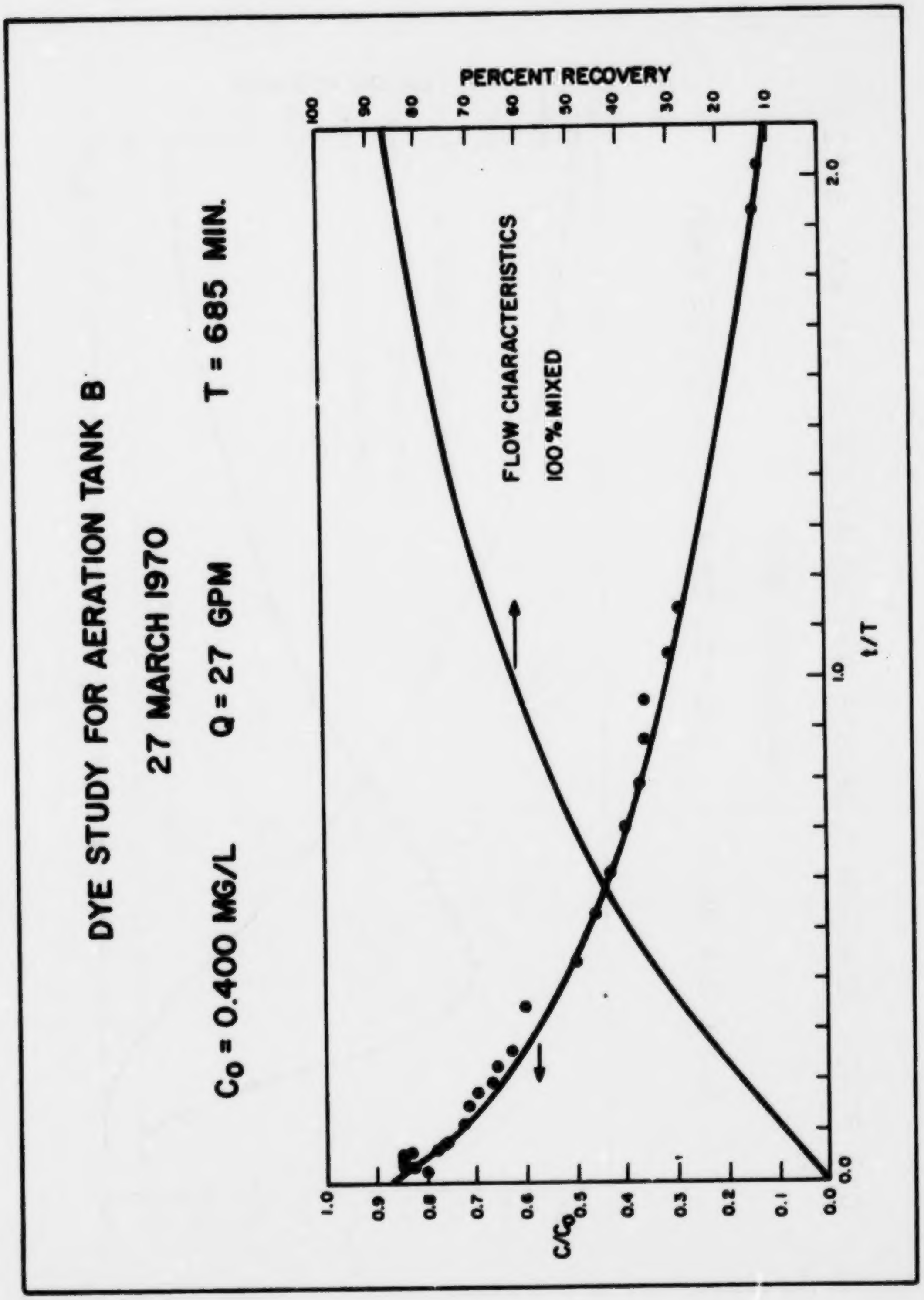

DYE STUDY FOR AERATION TANK B
27 MARCH 1970
C_O = 0.400 MG/L
Q = 27 GPM
T = 685 MIN.
PERCENT RECOVERY
100
90
80
70
60
50
40
30
20
10
FLOW CHARACTERISTICS
100% MIXED
C/C_O
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.0
1.0
2.0
t/T

Figure 70

DYE STUDY FOR AERATION TANK C

27 MARCH 1970

C_0 = 0.400 MG/L   Q = 23 GPM   T = 800 MIN.

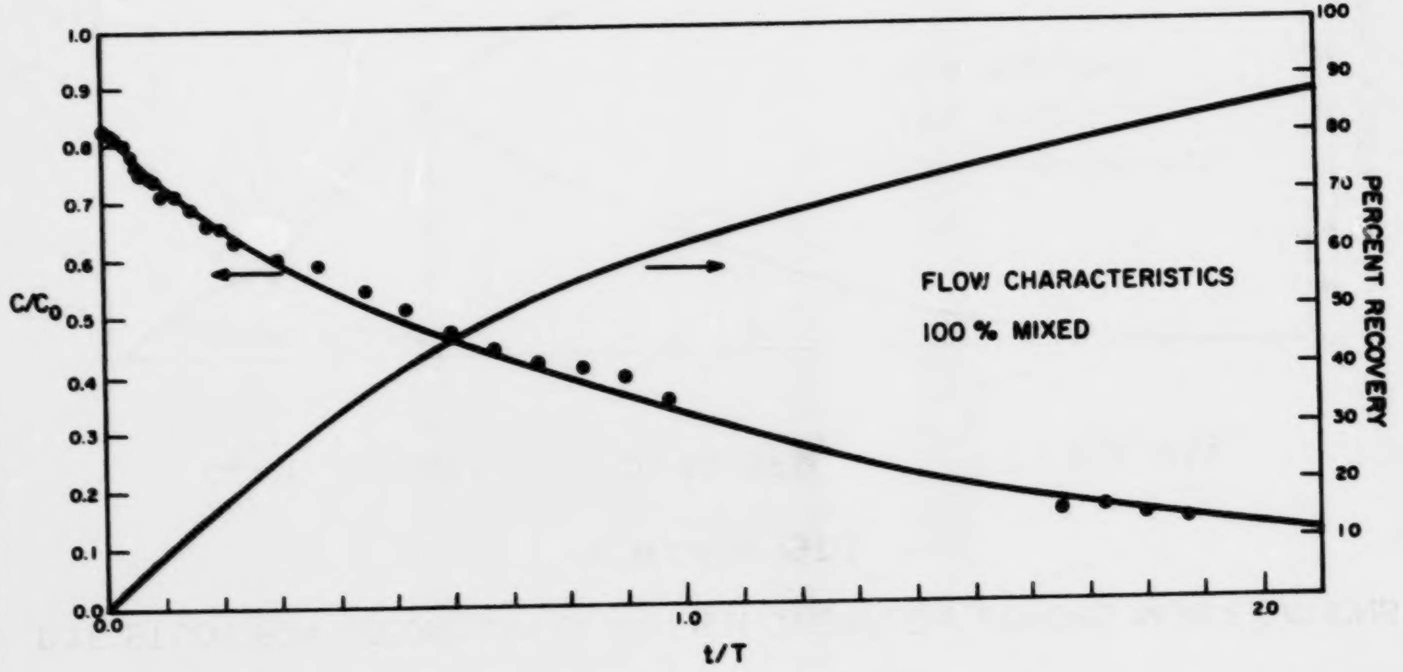


Figure 71

DYE STUDY FOR SECONDARY CLARIFIER OVERFLOW BEFORE MODIFICATIONS

4 MARCH 1970

C_0 = 0.21 MG/L   Q = 25 GPM   T = 202 MIN.

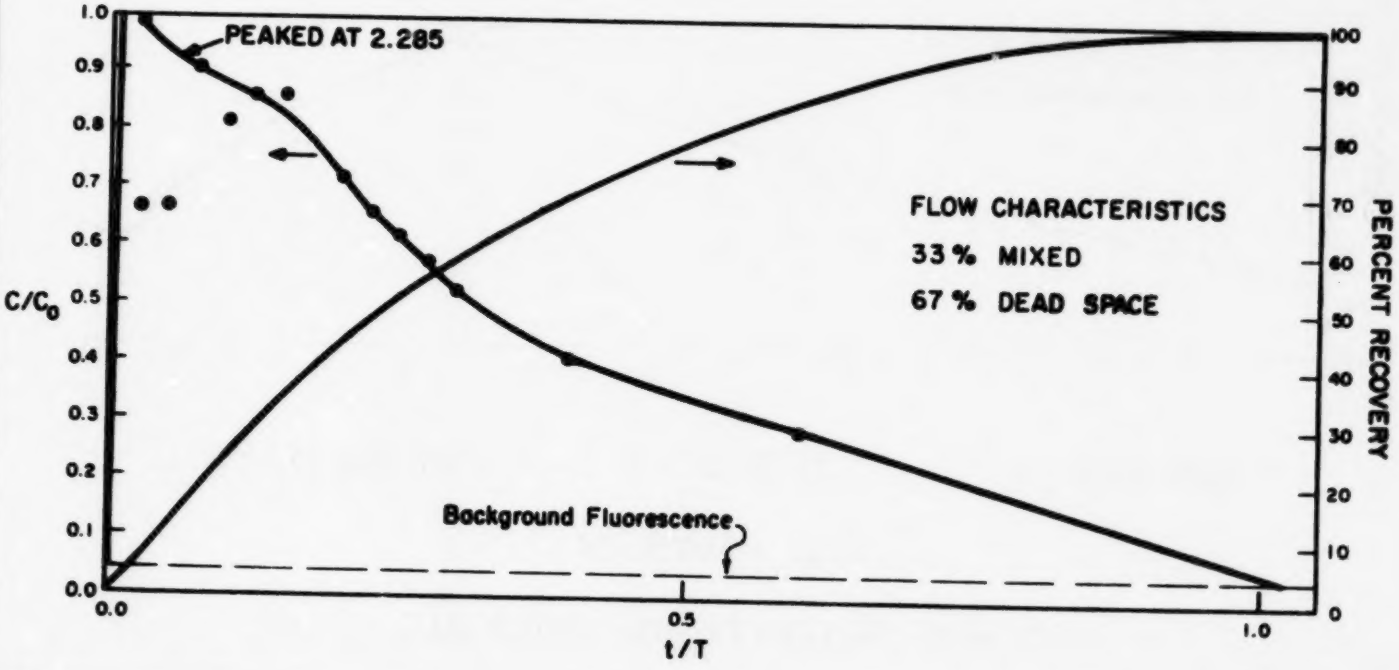


Figure 72

DYE STUDY FOR SECONDARY CLARIFIER UNDERFLOW BEFORE MODIFICATIONS

4 MARCH 1970

C_0 = 0.21 MG/L　　Q = 25 GPM　　T = 202 MIN.

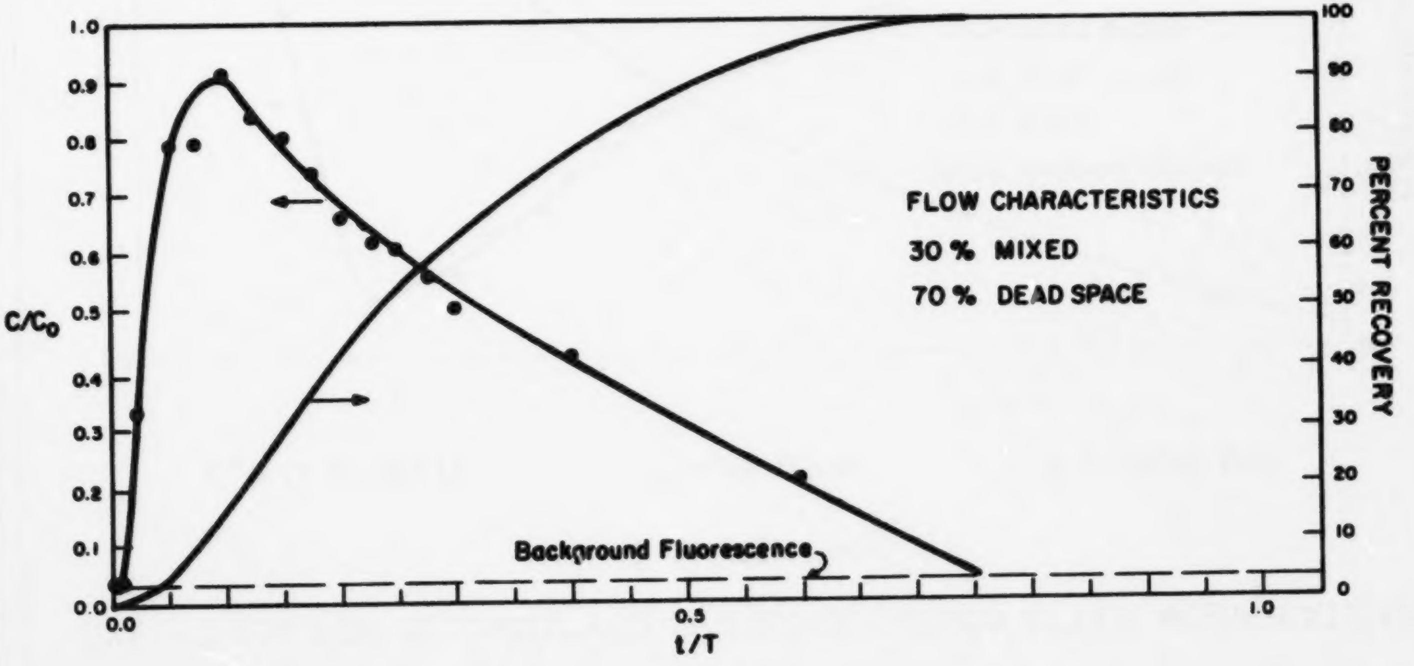
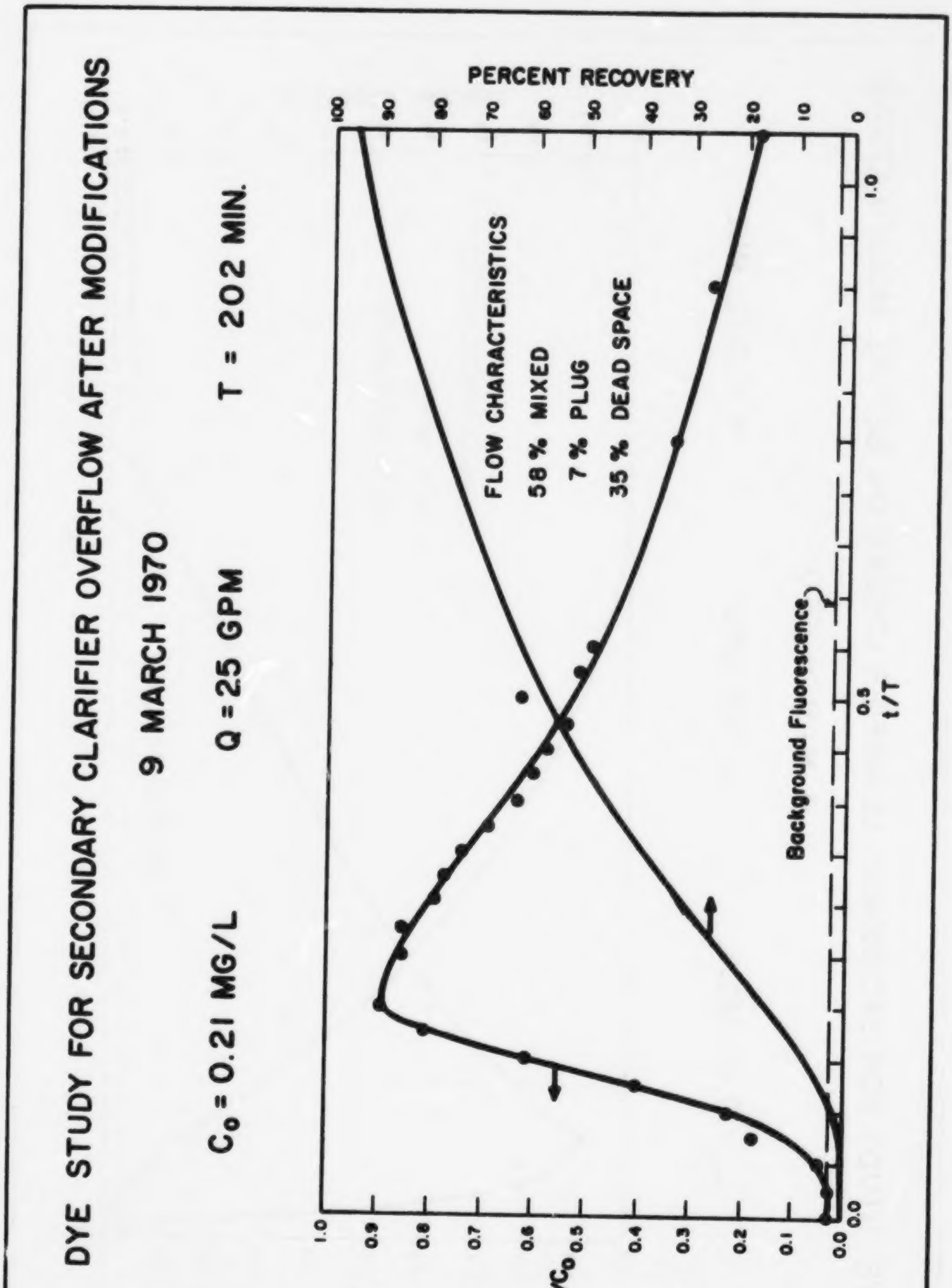


Figure 73


DYE STUDY FOR SECONDARY CLARIFIER OVERFLOW AFTER MODIFICATIONS

9 MARCH 1970

$C_o = 0.21$ MG/L $Q = 25$ GPM $T = 202$ MIN.

Figure 74

DYE STUDY FOR SECONDARY CLARIFIER UNDERFLOW AFTER MODIFICATIONS

9 MARCH 1970

C_0 = 0.21 MG/L Q = 25 GPM T = 202 MIN.

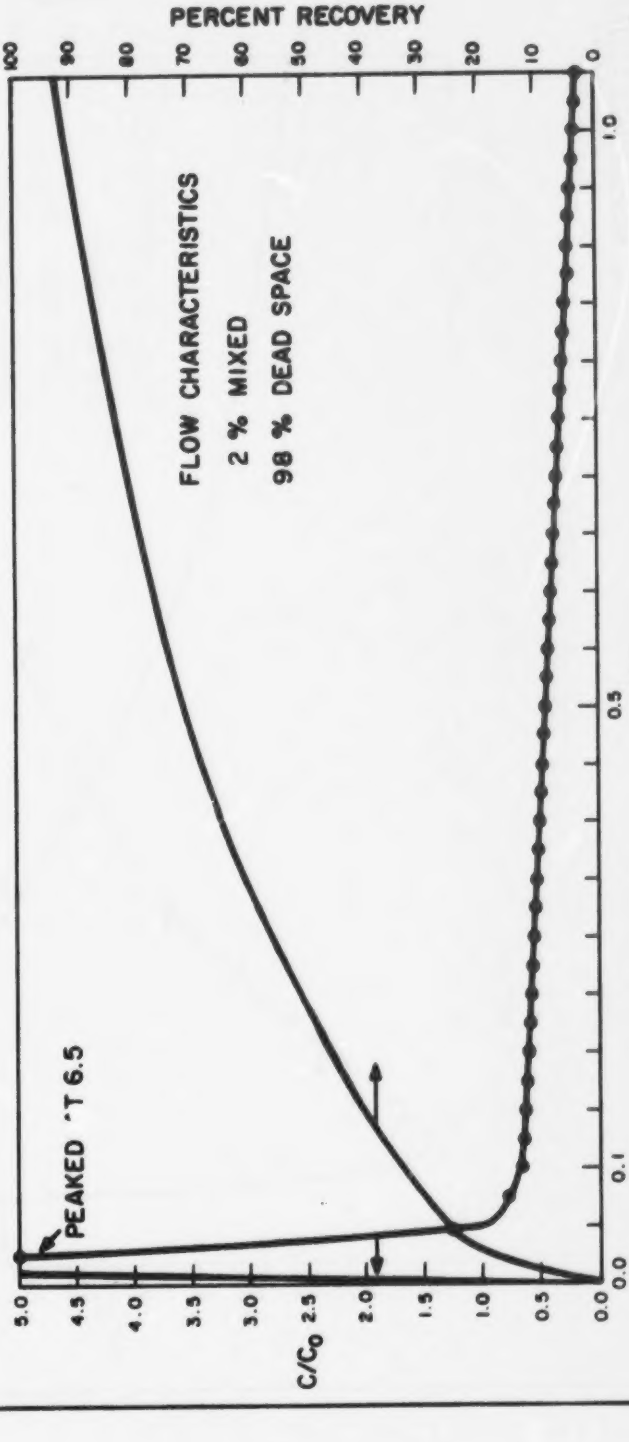
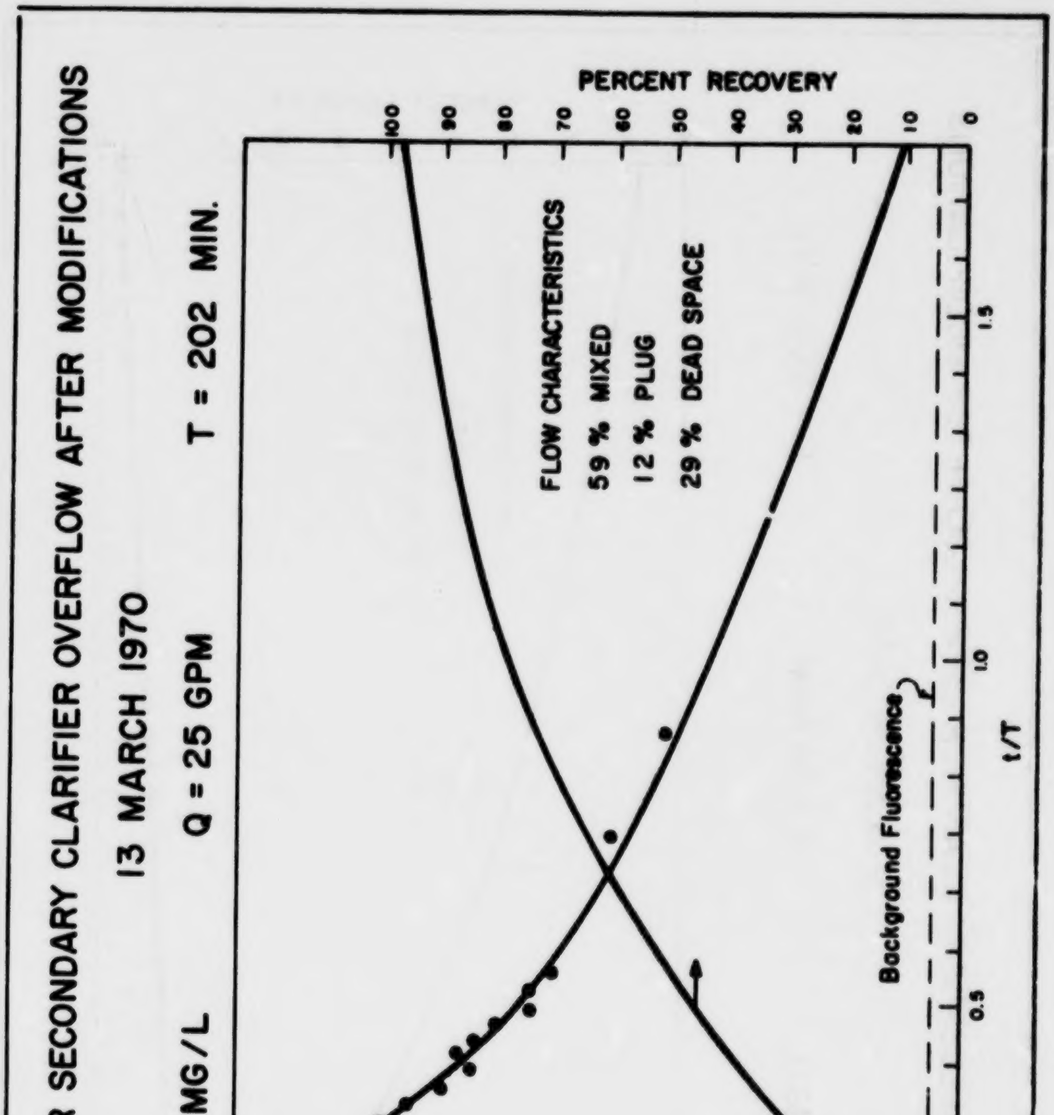


Figure 75



DYE STUDY FOR SECONDARY CLARIFIER OVERFLOW AFTER MODIFICATIONS

13 MARCH 1970

C_0 = 0.21 MG/L Q = 25 GPM T = 202 MIN.

Figure 76

DYE STUDY FOR SECONDARY CLARIFIER UNDERFLOW AFTER MODIFICATIONS

13 MARCH 1970

C_o = 0.21 MG/L Q = 5 GPM T = 1010 MIN.

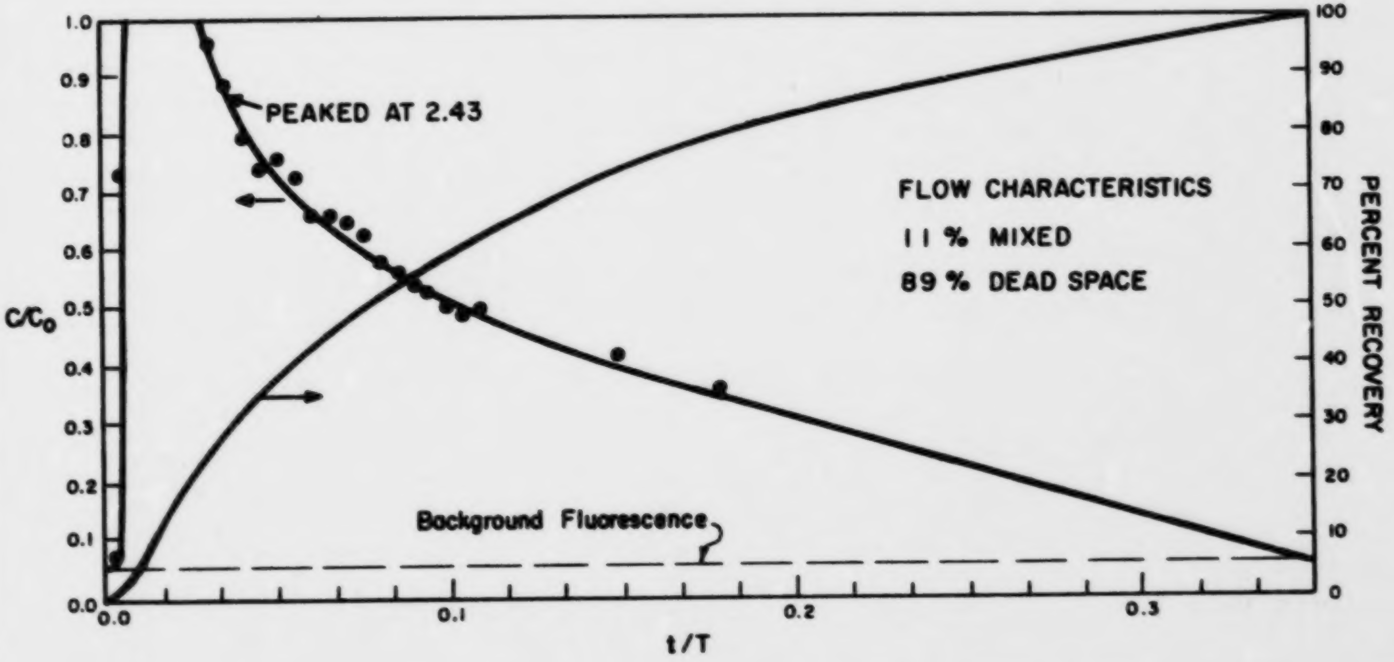


TABLE 25

SUMMARY OF DYE STUDY RESULTS AND FLOW CHARACTERISTICS

Process	Figure	Conditions		Dye	Flow Characteristics		
		Flow Rate	General	Recovery	Mixed	Plug	Dead Space
		GPM		Percent	Percent	Percent	Percent
Equalization	IV-7	45	Recycle at 350 gpm	80.0	77	--	23
Neutralization	IV-8	50	One stage	101.0	100	--	--
Neutralization	IV-9	50	Two stages	96.0	98	2	--
Reactor-Clarifier	IV-10	50	Turbine off	91.0	84	11	5
Reactor-Clarifier	IV-11	50	Turbine @ 25% speed	63.2	85	12	3
Aeration Tank B	IV-12	27	Diffused air	83.5	100	--	--
Aeration Tank C	IV-13	23	Mechanical aeration	85.0	100	--	--
Secondary Clarifier(a)	IV-14	50	Overflow at 25 gpm	38.0	33	--	67
Secondary Clarifier(a)	IV-15		Underflow at 25 gpm	30.0	30	--	70
Secondary Clarifier	IV-16	50	Overflow at 25 gpm	45.0	58	7	35
Secondary Clarifier	IV-17		Underflow at 25 gpm	54.0	2	--	98
Secondary Clarifier	IV-18	30	Overflow at 25 gpm	82.0	59	12	29
Secondary Clarifier	IV-19		Underflow at 5 gpm	13.0	11	--	89

(a) Before modifications to inlet and outlet structures.

the overflow and the fact that the tank had approximately the same amount of completely mixed space for both the overflow and the underflow.

Normally, a clarifier overflow would have at least a small amount of plug flow and the underflow would occupy only a very small fraction of the vessel. Investigation of the tank indicated that the inlet pipe was directed at the effluent structure, and that the effluent channel did not collect the overflow uniformly around the tank periphery.

Both secondary clarifiers were modified by installing a deflection plate over the inlet pipe and cutting more holes in the effluent channel. The dye studies were then repeated with two sets of conditions: the first with both the overflow and underflow at 25 gpm; and the second with the overflow at 25 gpm; and the underflow at five gpm. The results for both cases indicated considerable improvement in the hydraulic characteristics of the tanks. With the former conditions, the underflow used an indicated two percent of the volume of the tank and the overflow had seven percent plug flow characteristics. With the reduced underflow, the amount of plug flow for the overflow increased to 12 percent and the underflow used an insignificant 12 percent of the total volume of the clarifier.

Determination of Oxygenation Capacity

Although the oxygen transfer efficiency of aeration equipment is furnished by the manufacturers, this value is subject to many variables such as basin volume, basin geometry, nature of the wastewater, and environmental conditions. The purpose of this study, therefore, was to obtain general estimate of the oxygenation capacity of the operating system, using this information as appropriate in designing full scale facilities.

Procedure

The basic procedure applied in determining the oxygenation capacity of the system was the insitu approach as described by Kayser (Reference 2). This approach has the advantages of taking critical measurements with minimum interruption of plant operation and obtaining data under actual process conditions.

The variation of the oxygen content in an activated sludge system is expressed by the following equation:

$$\frac{dC}{dt} = K_L a \left(C_{sw} - C_s^*\right) - r \qquad \text{(VI-2)}$$

where:

$\frac{dC}{dt}$ = velocity of changing oxygen concentration (mg/l/hr)

K_La = overall mass transfer coefficient (hr^{-1})

C_{sw} = oxygen saturation under process conditions (mg/l)

C_s^* = oxygen saturation under equilibrium conditions (mg/l)

C = oxygen concentration in aeration basin (mg/l)

r = microbial respiration rate (mg/l/hr)

If wastewater and recycled sludge inputs to the aeration tank are stopped, the oxygen uptake rate will become relatively constant following 0.5 to 2 hours of aeration. When aeration is stopped, the oxygen content in the aeration tank decreases, the velocity of this decrease being the microbial respiration rate. If the aeration is started, the oxygen concentration will increase to a certain level and then remain constant (C^*_s). Once conditions are stabilized, i.e., $dC/dt = 0$ and r is constant, then K_La will be constant and can be evaluated by rearranging Equation (VI-2):

$$K_La = \frac{r}{C_{sw} - C^*_s} \qquad \text{(VI-3)}$$

The mass transfer coefficient, K_La, then must be corrected to a 20°C standard by:

$$K_La_{(T)} = K_La_{(20)}\ 1.024^{T-20} \qquad \text{(VI-4)}$$

The oxygenation capacity (O.C.) of the system can then be determined:

$$O.C. = K_La_{(20)}\ (C_{sw})\ (\text{tank volume}) \qquad \text{(VI-5)}$$

The transfer efficiency (T.E.) can then be calculated:

$$T.E. = \frac{O.C.}{\text{nameplate HP of aerator}} \qquad \text{(VI-6)}$$

Data Analysis

The test was performed using half of aeration basin "C" for the mechanical aerator evaluation. The wastewater and sludge return flow were stopped, the aerators were turned off following a brief aeration period, and the microbial respiration rate was determined using a galvanic cell oxygen analyzer. Two verification runs were made, the data being tabulated in Table 26. The oxygen saturation value of the wastewater was determined from parallel test tanks at similar environmental conditions. The oxygenation capacity and transfer efficiency values are calculated as follows:

Mechanical Aeration -

1. Mass Transfer Coefficient Determination

C_{sw} = 7.48 mg/l (observed)

C^*_s = 3.60 mg/l (observed)

T = 29°C

r avg. = 30 mg O_2/l/hr

$$K_La = \frac{r}{C_{sw} - C^*_s} = \frac{30}{3.88} = 7.74\ hr^{-1}$$

$$7.74 = K_La_{(20)}\ 1.024^{29-20}$$

$$K_La_{(20)} = \frac{7.74}{1.238} = 6.25\ hr^{-1}$$

2. Oxygenation Capacity Determination (20°C)

$$O.C. = \frac{6.25}{hr}\quad 7.48\ mg/l \quad 9{,}000\ gal. \quad \frac{8.34\ lbs}{10^6\ gal.\ mg/l}$$

$$O.C. = 3.50\ lbs\ O_2/hr$$

3. Transfer Efficiency Determination (20°C)

$$T.E. = \frac{3.50\ lbs\ O_2/hr}{1.5\ HP - 0.3\ HP\ (turbine)} = 2.90\ lbs\ O_2/HP\text{-}hr$$

TABLE 26

OXYGENATION CAPACITY DETERMINATION

Conditions: Nameplate H.P. - 1.5 (Mechanical Aeration)

Liquid Volume - 9,000 gal.

Temperature - 84°F = 29°C

Time (min.)	Analyzer Reading	Dissolved Oxygen (mg/l)
Run No. 1		
0:00	1.6	1.45
0:30	1.9	1.72
1:00	2.8	2.54
1:30	3.3	3.00
2:00	3.4	3.10
3:00	3.5	3.18
5:00	3.6	3.28

Avg. O_2 Uptake Rate, r, = 33 mg/l/hr

Time (min.)	Analyzer Reading	Dissolved Oxygen (mg/l)
Run No. 2		
0:00	1.6	1.45
1:00	2.0	1.82
2:00	2.6	2.46
3:00	3.0	2.73
4:00	3.4	3.04
5:00	3.5	3.19
6:00	3.6	3.38
7:00	3.6	3.28

Avg. O_2 Uptake Rate, r, = 28 mg/l/hr

Diffused Aeration -

1. **Mass Transfer Coefficient Determination**

$C_{sw} = 7.88$ mg/l (observed)

$C^*_s = 6.55$ mg/l (observed)

$T = 29^\circ C$

r avg. $= 8$ mg O_2/l /hr

$$K_La = \frac{r}{C_{sw} - C^*_s} = \frac{8}{1.33} = 6.0 \text{ hr}^{-1}$$

$$6.00 = K_La(20)\ 1.024^{29-20}$$

$$K_La(20) = \frac{6.00}{1.238} = 4.85 \text{ hr}^{-1}$$

2. **Oxygenation Capacity Determination** (20°C)

$$O.C. = \left(\frac{4.85}{hr}\right)\left(7.88 \text{ mg/l}\right)\left(18{,}000 \text{ gal.}\right)\left(\frac{8.34 \text{ lbs}}{10^6 \text{ gal. mg/l}}\right)$$

O.C. = 5.74 lbs O_2/hr

3. **Transfer Efficiency Determination** (20°C)

$$T.E. = \frac{5.74}{5 \text{ HP}} \text{ lbs } O_2/\text{hr} = 1.15 \text{ lbs } O_2/\text{HP-hr}$$

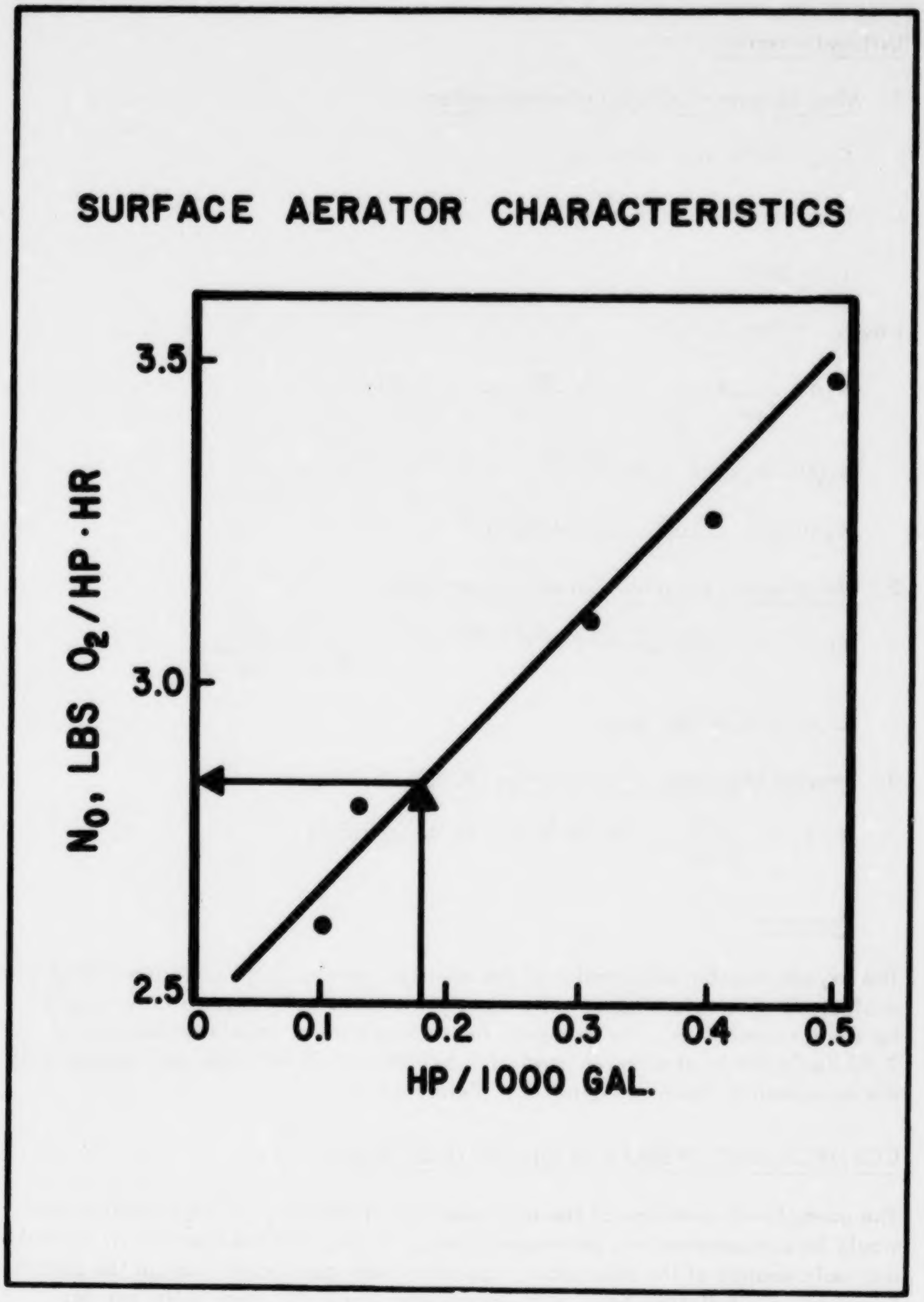
Summary

The oxygen transfer efficiencies of the aeration systems at the pilot plant have been evaluated. The values agree quite closely with observations previously reported for similar conditions. For example, the surface aerator transfer efficiency of 2.90 lbs O_2/HP hr at a power level of 1.5/9000 = 0.17 HP/1000 gal. agrees with the relationship shown in Figure 77. (Reference 3).

CONTROL AND OPERATION OF THE PILOT PLANT

The control and operation of the pilot plant involved many of the activities that would be encountered in a prototype system. To augment the operational control and maintenance of the pilot plant, operators were contracted through the duPont Company on a full-time basis with one operator per shift, three shifts per day,

Figure 77


SURFACE AERATOR CHARACTERISTICS
N₀, LBS O₂/HP·HR
3.5
3.0
2.5
0
0.1
0.2
0.3
0.4
0.5
HP/1000 GAL.

seven days per week. In addition, qualified laboratory technicians and the necessary analytical equipment were provided under this contract. Engineering-Science, Inc. directly supervised the operators and laboratory personnel resident at the plant site. Described herein is the wastewater collection and sampling system, and the pilot plant instrumentation and hydraulic controls that were utilized during the pilot plant study.

Wastewater Collection and Sampling Program

Transported Wastewater

The composition schedule for the integrated wastewater treated at the pilot plant was based on 1975 projected flow estimates as shown in Table 27. Based on these original flow estimates, approximately 44 percent of the wastewater was contributed by the participants outside of the duPont Chambers Works. This wastewater, 212,000 gallons per week as tabulated in Table 28, was transported to the pilot plant facility via tank truck on a five day per week basis. The trucking schedule is shown in Table 29. This schedule allowed for maximum utilization of the transport equipment while also satisfying the various truck loading requirements of the individual participants. Moreover, the schedule was arranged so that the compositing of the integrated wastewater was as close to field conditions as physically possible. The storage tanks at the pilot plant were operated in parallel on a continuous withdrawal basis. Each Monday the stored wastewater inventory was 82,000 gallons while on each Friday the stored wastewater inventory was a maximum of 164,000 gallons. This arrangement allowed for the continuous operation of the plant on a seven day per week basis using the integrated wastewater as a feed source.

The pilot plant studies were initiated on April 1, 1970. At that time, the B. F. Goodrich plant was still under construction and therefore no wastewater was transported from this participant until the Fall of 1970. The wastewater from Houdry Chemical Company was also omitted because of the small flow and because the analyses made during the bench scale studies indicated that most of the wastewater was uncontaminated once-through cooling water. As further revised flow estimates were made by the various participants, it became necessary to make adjustments to the wastewater compositing schedule. Tables 30 through 32 present these revised flow estimates and the revised trucking schedule for the "winter loading" conditions at the pilot plant.

The truck loading facilities at each of the various participant locations were the same sampling points utilized during the bench scale wastewater characterization studies. The transport trailers were equipped with 400 gpm gasoline-powered, self-priming centrifugal pumps for self loading. Each loading facility was fitted

TABLE 27

INDIVIDUAL PARTICIPANT CONTRIBUTIONS FOR THE INTEGRATED PILOT PLANT WASTEWATER SUMMER LOADINGS

Participant	Estimated 1975 Flow (MGD)*	Percent of Total	Gal. Needed Per Day	Gal. Needed Per Week	Number of 5600 Gal. Tank Truck Loads/Wk.
duPont Chambers Works	45.21	56.06	40,363	282,542	(Pumped to Pilot Plant)
Texaco	5.40	6.70	4,824	33,768	6
Shell	3.00	3.70	2,678	18,749	3
Mobil	14.00	17.36	12,992	87,749	16
Hercules	0.14	0.17	122	856	856 Gal.
duPont Repauno	0.29	0.36	259	1,814	1,814 Gal.
Monsanto	3.00	3.72	2,678	18,748	3
B.F. Goodrich**	1.20	1.49	1,073	7,509	—
du Pont Carney's Point	2.40	2.98	2,146	15,019	3
Municipalities	6.00	7.44	5,357	37,497	7
TOTAL	80.64	100.00%	72,492	503,996	

*Estimates were effective as of May 1970

**Plant was not in operation during the summer

TABLE 28

PARTICIPANT WASTEWATER CONTRIBUTIONS BASED ON TRUCKING SCHEDULE*

Participant	Number of Truck Loads Per Week (5600 Gal. ea.)	Partial Loads Per Week (gal.)	Total Gallons Per Week	Percent of Total as Trucked
duPont Chambers Works	(Pumped to Pilot Plant)	-	292,000	57.94
Texaco	6	-	33,600	6.67
Shell	3	-	16,800	3.33
Mobil	12	19,000	86,200	17.10
Hercules	-	750	750	0.15
duPont Repauno	-	1,900	1,900	0.37
Monsanto	3	-	16,800	3.33
duPont Carney's Point	3	-	16,800	3.33
Municipalities	7	-	39,200	7.78
TOTAL			504,050	100.00%

*For summer loading

TABLE 29

DEEPWATER PILOT PLANT TRUCKING SCHEDULE FOR SUMMER LOADINGS

Monday	Tuesday	Wednesday	Thursday	Friday
Number One Truck				
Mobil	Mobil	Mobil	Mobil	Texaco
Texaco	Mobil	Mobil	Mobil	Mobil - 4650 gal.
Monsanto	Mobil–4650 gal.	Monsanto	Municipalities	Repauno - 950 gal.
Municipalities	Repauno–950 gal.	Municipalities	Municipalities	Monsanto
	Municipalities	Municipalities		Municipalities
Number Two Truck				
Shell	Mobil - 5225 gal.	Texaco	Texaco	Mobil - 5225 gal.
Carney's Point	Hercules - 375 gal.	Shell	Texaco	Hercules - 375 gal.
Mobil	Texaco	Carney's Point	Mobil	Carney's Point
Mobil	Mobil			Shell
				Mobil

Capacity for each truck = 5600 gal.

TABLE 30

INDIVIDUAL PARTICIPANT CONTRIBUTIONS FOR INTEGRATED PILOT PLANT WASTEWATER WINTER LOADINGS

Participant	Revised 1975 Flow (MGD)*	Percent of Total	Gal. Needed Per Day	Gal. Needed Per Week	Number of 5600 Gal. Tank Truck Loads/Wk.
duPont Chambers Works	45.21	54.50	39,240	274,680	(Pumped to Pilot Plant)
Texaco	6.80	8.20	5,904	41,328	7
Shell	3.00	3.61	2,599	18,194	3
Mobil	14.00	16.87	12,146	85,024	15
Hercules	0.16	0.20	144	1,008	1,008 Gal.
duPont Repauno	0.25	0.30	216	1,512	1,512 Gal.
Monsanto	3.25	3.92	2,822	19,757	3
B.F.Goodrich	1.30	1.57	1,130	7,912	7,912 Gal.
duPont Carney's Point	3.18	3.83	2,758	19,303	3
Houdry**	0.25	0.30	216	1,512	--
Municipalities	5.57	6.70	4,824	33,768	6
TOTAL	82.97 MGD	100.00%	71,990	503,998	

*Estimates were effective as of August 1970

** Waste not trucked to Pilot Plant

TABLE 31

PARTICIPANT WASTEWATER CONTRIBUTIONS BASED ON REVISED TRUCKING SCHEDULE*

Participant	Number of Truck Loads Per Week (5600 Gal. ea.)	Partial Loads Per Week (gal.)	Total Gallons Per Week	Percent of Total
duPont Chambers Works	—	—	288,800	57.30
Texaco	7	—	39,200	7.78
Shell	3	—	16,800	3.33
Mobil	11	19,900	81,500	16.17
Hercules	—	1,000	1,000	0.20
duPont Repauno	—	1,500	1,500	0.30
Monsanto	3	—	16,800	3.33
B. F. Goodrich	—	8,000	8,000	1.59
duPont Carney's Point	3	—	16,800	3.33
Municipalities	6	—	33,600	6.67
TOTAL			504,000	100.00

* For winter loadings

TABLE 32

DEEPWATER PILOT PLANT TRUCKING SCHEDULE FOR WINTER LOADINGS

Number One Truck

Monday	Tuesday	Wednesday	Thursday	Friday
Mobil	Mobil	Mobil	Mobil	Texaco
Texaco	B.F.Goodrich- 4,000 gal.	Mobil	Mobil	B.F. Goodrich- 4,000 gal.
Monsanto		Monsanto	Municipalities	
Municipalities	Mobil - 4850 gal. Repauno -750 gal.	Municipalities Municipalities	Municipalities	Mobil - 4850 gal. Repauno - 750 gal.
	Municipalities			Monsanto

Number Two Truck

Monday	Tuesday	Wednesday	Thursday	Friday
Shell	Mobil - 5100 gal.	Texaco	Texaco	Mobil - 5100 gal.
Carney's Point	Hercules - 500 gal.	Texaco	Texaco	Hercules - 500 gal.
Mobil	Texaco	Shell	Mobil	Carney's Point
Mobil	Mobil	Carney's Point		Shell
				Mobil

Capacity for each truck = 5600 gal.

with three inch pipe connections and located to allow access to trucking equipment. In some cases, arrangements were made to fill the tank trucks by in-house pumping facilities. All the truck loads represented grab type samples with the exception of the duPont Repauno and the Hercules samples. These two plants had equalization and/or large volume compositing facilities in-house.

Chambers Works Wastewater Collection System

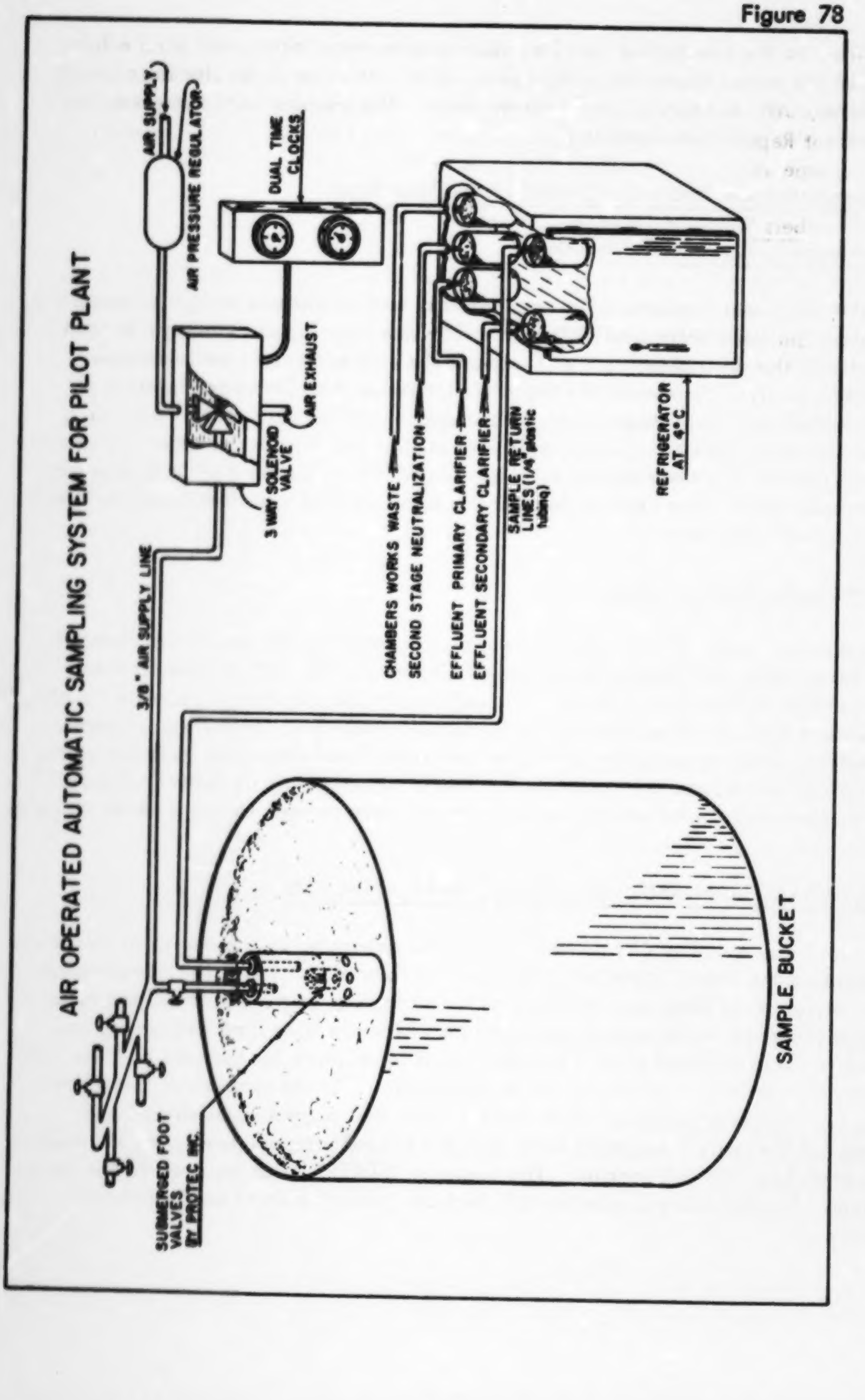
The wastewater conveyance system at the Chambers Works consisted of two streams-namely, the organic waste stream and the cooling water waste stream. The organic waste stream was utilized for the pilot plant make-up. This wastewater was pumped directly to the pilot plant on a continuous basis through a flow metering and control system. A composite sampling system as described later in this Section was used to collect 24-hour composite samples.

Modifications were later made to this collection system because of necessary construction carried out by the duPont Company at the location of the pump intake. A 1,500 gallon head tank was installed and became the intake facility for the wastewater pump. The organic waste stream was then transferred via two additional pumps to the head tank from the two streams that made up the total organic waste stream flow.

Pilot Plant Sampling System

In order to obtain selected composite samples for the evaluation of unit processes, an automatic sampling system was installed at the pilot plant. Since the flow through the plant was constant, grab type samples taken on a regular sequence and composited over identical time periods represented true composites proportional to flow. The system itself, as shown in Figure 78, consisted of six Protec Model sampling foot valves connected in parallel with an electrically controlled air supply system. The foot valves were submerged in the integrated wastewater sampling bucket, the Chambers Works sampling bucket, the second stage neutralization tank, the effluent weir box of the primary clarifier, and the effluent weir boxes of the two final clarifiers. The force of the water filled the 20 milliliter sample chamber via a ball check valve. When the timing clocks actuated the three-way solenoid valve, air pressure was applied to the sampling chambers forcing the sample out the effluent pipe of the samplers and to compositing carboys in a refrigerator at 4 °C. The cycle time for each grab sample was set for 15 minute intervals with 25 seconds of air pressure applied per interval. Using this system, a sample of approximately two liters per 24 hours was collected at each sample point. Once every 24-hour period, the carboys were transported to the laboratory for analysis.

Figure 78


AIR OPERATED AUTOMATIC SAMPLING SYSTEM FOR PILOT PLANT

AIR SUPPLY
AIR PRESSURE REGULATOR
DUAL TIME CLOCKS
AIR EXHAUST
3 WAY SOLENOID VALVE
3/8" AIR SUPPLY LINE
CHAMBERS WORKS WASTE
SECOND STAGE NEUTRALIZATION
EFFLUENT PRIMARY CLARIFIER
EFFLUENT SECONDARY CLARIFIER
SAMPLE RETURN LINES (1/4" plastic tubing)
REFRIGERATOR AT 4°C
SUBMERGED FOOT VALVES BY PROTEC INC.
SAMPLE BUCKET

In addition to the composited samples, grab samples were taken once per 24-hour period of the mixed liquor and sludge blowdowns. As these particular samples are time-dependent, composites could not be made. The samples from each tank truck load were also grab type samples.

Instrumentation and Hydraulic Control of the Pilot Plant

Instrumentation of the Pilot Plant

The pilot plant was equipped with only minimal control instrumentation as complete laboratory facilities were available at the Chambers Works, alleviating the need for sophisticated instrumentation at the plant site. The control instrumentation consisted of a dual Honeywell pH recorder-controller with Universal Interloc pH probes installed in both stages of the two stage neutralization system. Recording instrumentation consisted of Honeywell temperature and dissolved oxygen equipment that was installed in the aeration tankage. In addition, portable pH and dissolved oxygen instruments were kept on hand at the pilot plant to spot check and calibrate the recording instrument.

Hydraulic Control System

The hydraulic control of the pilot plant was augmented by the use of two Hammel Dahl Flow Tubes with Honeywell recorders located on the total process flow and the Chambers Works waste streams. Manually-operated diaphragm valves were placed downstream from the flow tubes and provided adequate flow controlling schemes. In addition, Sparling propeller-type flow indicator/totalizers were installed on the feed system to the aeration tanks and the sludge return system as noted in Figure 77. Each sludge return meter was downstream from in-line screens to avoid meter plugging.

DATA COLLECTION AND ELECTRONIC DATA ANALYSIS

The voluminous quantity of information that was generated in the course of this study necessitated the use of computer techniques for data processing. The development of the design parameters and coefficients for biological waste treatment processes involves numerous mathematical calculations which are both time-consuming and subject to computational error. In addition, it is valuable for the user of these data to know the statistical reliability of his information. In recognition of these limitations, two computer programs were written. The first program summarized and printed out the daily pilot plant data, and the second program developed the design parameters from this information. The acronym "STATPK" was assigned to the latter program. The following discussion will include a description of each of these programs.

Data Collection and Management Procedures

An analytical program was established around each of the unit processes at the pilot plant and designed for maximum data retrieval and utilization. Since computer techniques were used to summarize and tabulate the data, it was convenient to identify each sample by an eleven digit number as follows:

***	**	**	**	**
LOCATION	YEAR	MONTH	DAY	HOUR

The location number of the individual samples are outlined in Table 33. The hour code was based on the 24-hour system and designated either the time of grab sampling or the end of the compositing period.

Samples were transported to the laboratory each morning and the analytical schedule presented in Table 34 was followed. Results of analytical tests such as oxygen uptake, solids and COD were sent back to the pilot plant during the afternoon of the sampling day. This procedure provided direct operational control of the pilot plant based on these laboratory results.

Data Summary Computer Program

The basic procedure for handling the raw laboratory data included its tabulation on standardized data sheets, transferring it to computer cards, and processing it using a FORTRAN program. This basic procedure was used successfully to handle the data from the wastewater characterization and bench scale treatability studies. The Fortran program as described in Section V was modified to read out the pilot plant responses to various wastewater inputs. The output sheets from this program summarized the data from each of the unit processes and presented all necessary parameters of the operation unit. Additionally, the total pilot plant performance was presented in terms of removal efficiencies across the plant. This program was run at the end of each calendar month and printouts were presented as monthly task reports.

STATPK Program

The availability and utility of high-speed electronic computers gives the environmental engineer a tool which he can use to relieve himself of tedious and complicated mathematical procedures. In view of the myriad of data accumulated during the bench and pilot scale phases of this project, a computer program was developed to perform the necessary mathematical operations on biological waste treatment process information and to arrive at the required design information and the errors associated with it. The resolution of the pilot plant data is subsequently presented in this Section. A description of the STATPK program is presented in Appendix A.

TABLE 33

IDENTIFICATION AND LOCATION OF SAMPLE POINT NUMBERS

Description	Number
Plant Inlet – Raw Wastewater	601
Neutralization Process	610
Effluent from Second Stage Neutralization	610
Chemical Feed to Neutralization	613
Primary Clarification Process	620
Effluent from Primary Clarification	621
Chemical Feed to Primary Clarifier	622
Sludge from Primary Clarifier	623
Activated Sludge Process	630
Mixed Liquor from Aeration Tank A	631
Mixed Liquor from Aeration Tank B	632
Mixed Liquor from Aeration Tank C	633
Filtered Effluent from Activated Sludge Process	634
Settled Effluent from Activated Sludge Process	636
Waste Sludge from Activated Sludge Process	637
Return Sludge to Activated Sludge Process	638
Final Effluent from Pilot Plant	699

TABLE 34

DAILY ANALYTICAL SCHEDULE FOR PILOT PLANT

	601	610	621	623	631	632	633	634	636	637	638	699	Truck Samples
Alkalinity													
Acidity													
Neut.w/acid to 7													
Neut.w/base to 7													
TDS	x								x				
VDS	x								x				
TSS	x	x	x	x	x	x	x		x	x	x		
VSS	x	x	x	x	x	x	x		x	x	x		
COD unfiltered	x		x						x				x
COD filtered	x		x		x	x	x	x					
BOD_5 unfiltered	x		x						x				
BOD_5 filtered	x		x					x					
TOC unfiltered	x		x						x				
TOC unfiltered								x					
TOD unfiltered	x		x						x				x
TOD filtered								x					x
pH	x	x	x	x	x	x	x		x				
Elec. Cond.													
Kjeldahl N.			x									x	
Ammonia N.			x									x	
$NO_2 + NO_3$			x									x	
Total P			x									x	
Phenol	x											x	
MBAS	x											x	
Color	x		x									x	
Grease and Oil	x											x	
Heavy Metals	x		x									x	x
Volume				F						F			F
Flow	F										F		
Lime added (ft.)		F											
Lime Sol. (#/gal.)		F											
Temp. (°F)	x				x	x	x					x	
0_2 Uptake					x	x	x						
SVI					x	x	x						

x = Lab Analysis
F = Field Data

PILOT PLANT PROCESS EVALUATION - BIOLOGICAL TREATMENT

The pilot plant process evaluation with respect to the biological removal of organics was conducted in a manner similar to that previously described for the bench scale portion of Section V. The basic approach involved the application of various organic loadings to the activated sludge system while monitoring the resultant responses in terms of sludge build-up, organic removal efficiency, and oxygen utilization. In order to further delineate this evaluation, the organic loading levels were applied under both summer and winter conditions. Therefore, the hydraulic and organic loadings could be controlled with some semblance of temperature regulation.

The intent of this section is to describe the operating schedule followed during this process evaluation, present the summarized results of the data gathered during these tests, define the design parameters and coefficients as developed from a statistical analysis of the data, and discuss the effects of temperature and transient loading on the biological system. As the abundance of data generated during the pilot plant studies prevents its total inclusion in this text, only pertinent data are presented. The daily operational data has been presented under separate cover as monthly task reports.

Operating Schedule

The pilot plant operating schedule as originally envisioned is schematically outlined in Figure 79. This schedule was generally implemented throughout the pilot studies with the following exceptions:

(a) the proposed organic loadings of 0.75 and 1.2 lbs BOD_5/lb MLVSS/day were never obtained in September and October of 1970 as the cooling water usage of the various participants resulted in a lower than anticipated BOD concentration in the untreated wastewater; and,

(b) the initiation of the winter loading studies was delayed by a trucking strike which occurred in November of 1970.

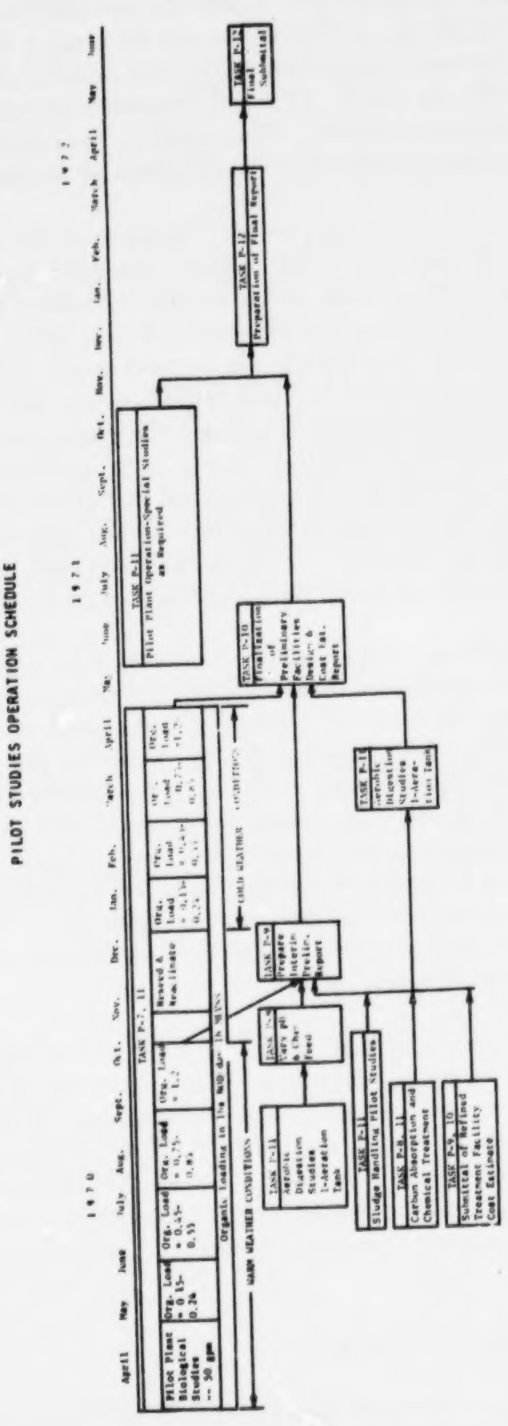
These two combined factors forced a scheduling change which substituted the transient loading study for the high organic loading study. Moreover, extremely cold weather resulted in a two week shutdown of the pilot plant during February of 1971. Ancillary process evaluation studies such as carbon adsorption, sludge handling, and filtration were expanded to include necessary design and treatment process evaluation.

Results of the Summer and Winter Loading Conditions

General

The pilot scale biological treatment data covering the activated sludge studies are

Figure 79


PILOT STUDIES OPERATION SCHEDULE
1970
1971
1972
April
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.
Feb.
March
April
May
June
July
Aug.
Sept.
Oct.
Nov.
Dec.
Jan.
Feb.
March
April
May
June
TASK P-7, 11
Pilot Plant Biological Studies -- 50 gpm
Org. Load = 0.15-0.24
Org. Load = 0.45-0.55
Org. Load = 0.75-0.85
Org. Load = 1.2
Reseed & Reacclimate
Org. Load = 0.15-0.24
Org. Load = 0.45-0.55
Org. Load = 0.75-0.85
Org. Load = 1.2
Organic Loading in lbs BOD day lb MLVSS
WARM WEATHER CONDITIONS
COLD WEATHER CONDITIONS
TASK P-11
Aerobic Digestion Studies 1-Aeration Tank
TASK P-8
Vary pH & Chem. Feed
TASK P-9
Prepare Interim Prelim. Report
TASK P-11
Sludge Handling Pilot Studies
TASK P-8, 11
Carbon Absorption and Chemical Treatment
TASK P-9, 10
Submittal of Refined Treatment Facility Cost Estimate
TASK P-11
Aerobic Digestion Studies 1-Aeration Tank
TASK P-10
Finalization of Preliminary Facilities Design & Cost Est. Report
TASK P-11
Pilot Plant Operation-Special Studies as Required
TASK P-12
Preparation of Final Report
TASK P-12
Final Submittal

presented graphically in Figures 80 through 83. Figure 80 presents the organic loadings in terms of COD and BOD_5 while Figures 81 and 82 present the COD and BOD_5 concentrations of the untreated wastewater (601), the primary clarifier effluent (621), and the final effluent (636). Figure 83 presents the mixed liquor solids (MLSS) concentrations and temperature. Additionally, operational modes are noted on each of the Figures in conjunction with explanations of process difficulties.

At the outset, several general conclusions can be established from the pilot plant data. First, there is a distinct seasonal variation in the organic concentration of the combined wastewater. This variation is underscored by the difference in the COD and BOD_5 values of the raw wastewater during May and June of 1970. The average BOD_5 of the raw wastewater during May was approximately 350 mg/l, while during the last of June the average BOD_5 was approximately 200 mg/l. Low BOD_5 and COD values were experienced throughout the warm summer months. This significant variation is the result of additional usage of once-through cooling water during the summer months, acting as a diluent to the raw wastewater. As economic considerations dictate the in-plant segregation of cooling and process wastewaters, it is expected that this seasonal variation of the organic characteristics in the regional system will be less pronounced than experienced at the pilot plant.

In addition to the seasonal variations, daily organic variations were also experienced. The changing nature of the wastewaters which were both trucked from the participating industries as well as pumped directly from the Chambers Works plant resulted in a restricted form of transient loading to the aeration basin. Based on participant equalization requirements and the equalization of flows in the interceptor, it is anticipated that the degree of fluctuation in organic loading will not be any more severe in the full scale system than observed in the pilot plant studies. However, as a precautionary measure, more pronounced fluctuations were deliberately imposed on the pilot plant system using the Chambers Works wastewater. The results are subsequently discussed in this section, although no marked deterioration of the biological system was noted during this test series.

Several minor biological upsets were observed during the pilot plant studies as a result of sulfide dumps, short-lived pH variations, and nitro-benzene dumps. Although the removal efficiencies were reduced during these upsets, a complete biological kill was never experienced during the entire pilot plant operation. This notwithstanding, a biological system is subject to occasional physical, chemical, biochemical, or environmental stresses which temporarily reduce the overall system efficiency. Proper design features, however, can minimize biological upsets.

Biological Treatment Removal Efficiencies

The observed removal efficiencies in terms of BOD_5 and COD generally decrease

Figure 80

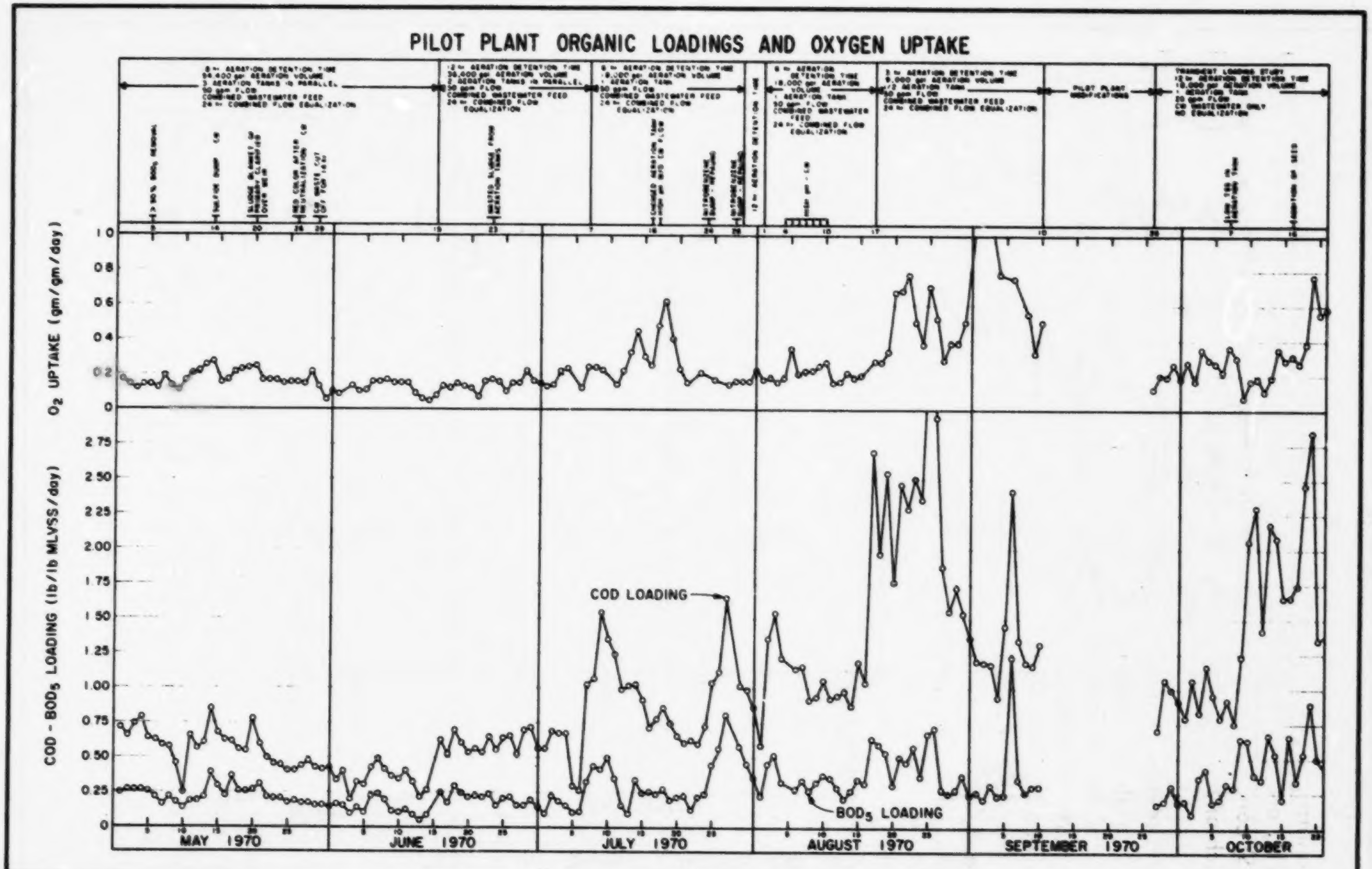

PILOT PLANT ORGANIC LOADINGS AND OXYGEN UPTAKE
8 hr AERATION DETENTION TIME
54,400 gal AERATION VOLUME
3 AERATION TANKS IN PARALLEL
50 gpm FLOW
COMBINED WASTEWATER FEED
24 hr COMBINED FLOW EQUALIZATION
12 hr AERATION DETENTION TIME
36,400 gal AERATION VOLUME
2 AERATION TANKS IN PARALLEL
50 gpm FLOW
COMBINED WASTEWATER FEED
24 hr COMBINED FLOW EQUALIZATION
6 hr AERATION DETENTION TIME
18,000 gal AERATION VOLUME
1 AERATION TANK
50 gpm FLOW
COMBINED WASTEWATER FEED
24 hr COMBINED FLOW EQUALIZATION
12 hr AERATION DETENTION TIME
6 hr AERATION DETENTION TIME
18,000 gal AERATION VOLUME
1 AERATION TANK
50 gpm FLOW
COMBINED WASTEWATER FEED
24 hr COMBINED FLOW EQUALIZATION
3 hr AERATION DETENTION TIME
9,000 gal AERATION VOLUME
1/2 AERATION TANK
50 gpm FLOW
COMBINED WASTEWATER FEED
24 hr COMBINED FLOW EQUALIZATION
PILOT PLANT MODIFICATIONS
TRANSIENT LOADING STUDY
12 hr AERATION DETENTION TIME
18,000 gal AERATION VOLUME
1 AERATION TANK
25 gpm FLOW
CB WASTEWATER ONLY
NO EQUALIZATION
O2 UPTAKE (gm/gm/day)
1.0
0.8
0.6
0.4
0.2
0
COD - BOD5 LOADING (lb/lb MLVSS/day)
2.75
2.50
2.25
2.00
1.75
1.50
1.25
1.00
0.75
0.50
0.25
0
COD LOADING
BOD5 LOADING
MAY 1970
JUNE 1970
JULY 1970
AUGUST 1970
SEPTEMBER 1970
OCTOBER

Figure 80
(Continued)

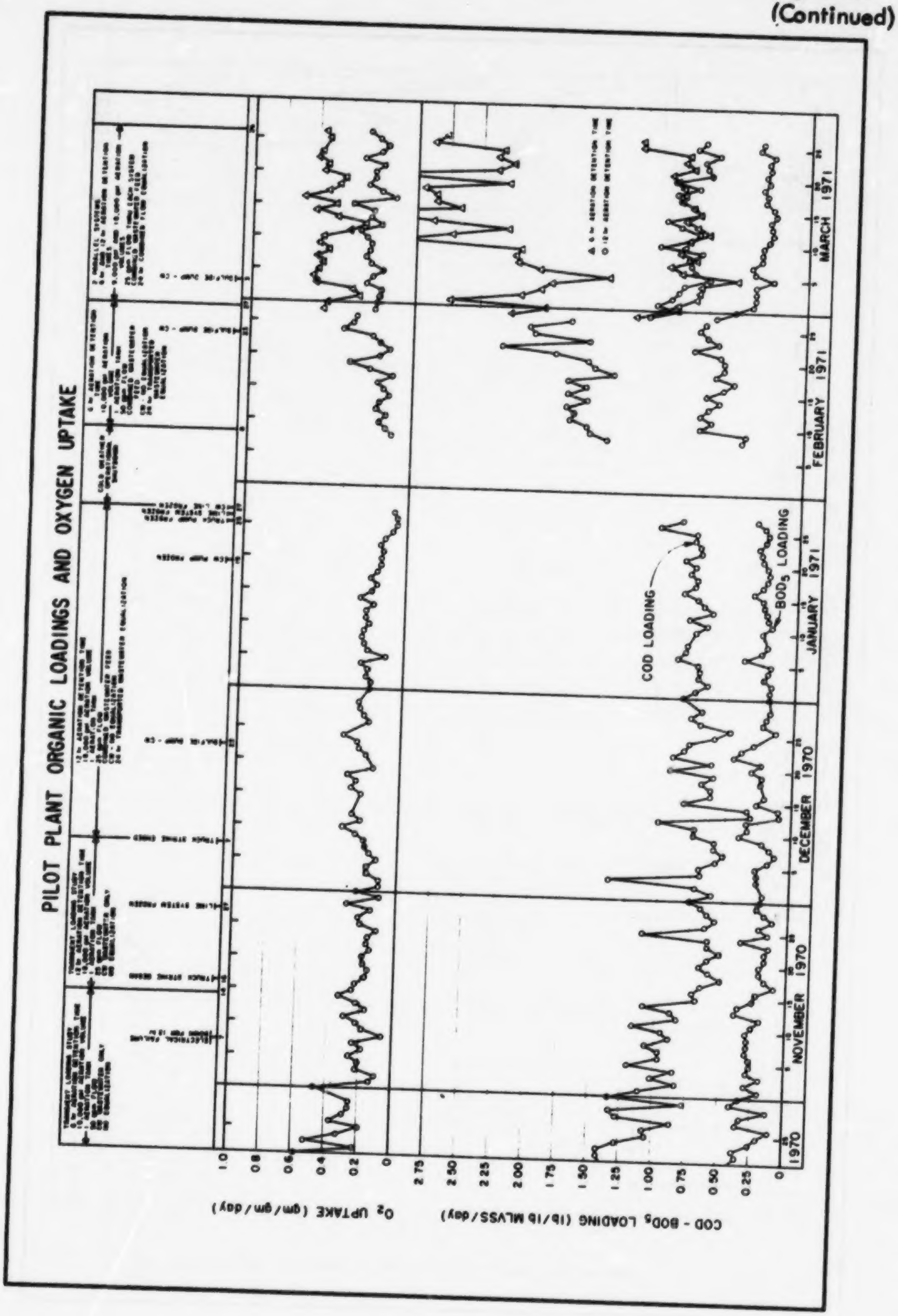


Figure 81

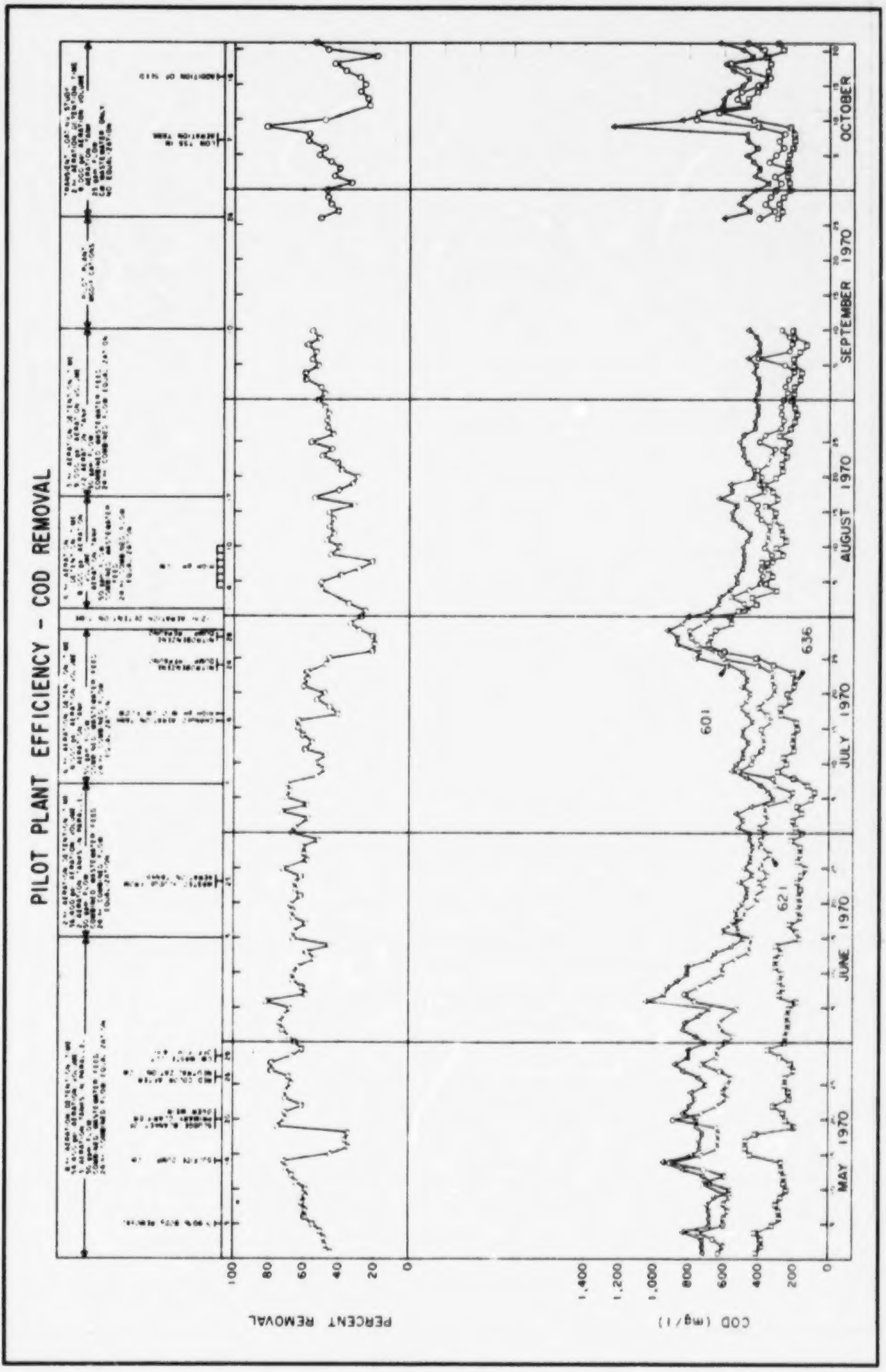

PILOT PLANT EFFICIENCY - COD REMOVAL
PERCENT REMOVAL
100
80
60
40
20
0
COD (mg/l)
1,400
1,200
1,000
800
600
400
200
0
601
621
636
MAY 1970
JUNE 1970
JULY 1970
AUGUST 1970
SEPTEMBER 1970
OCTOBER

Figure 81
continued

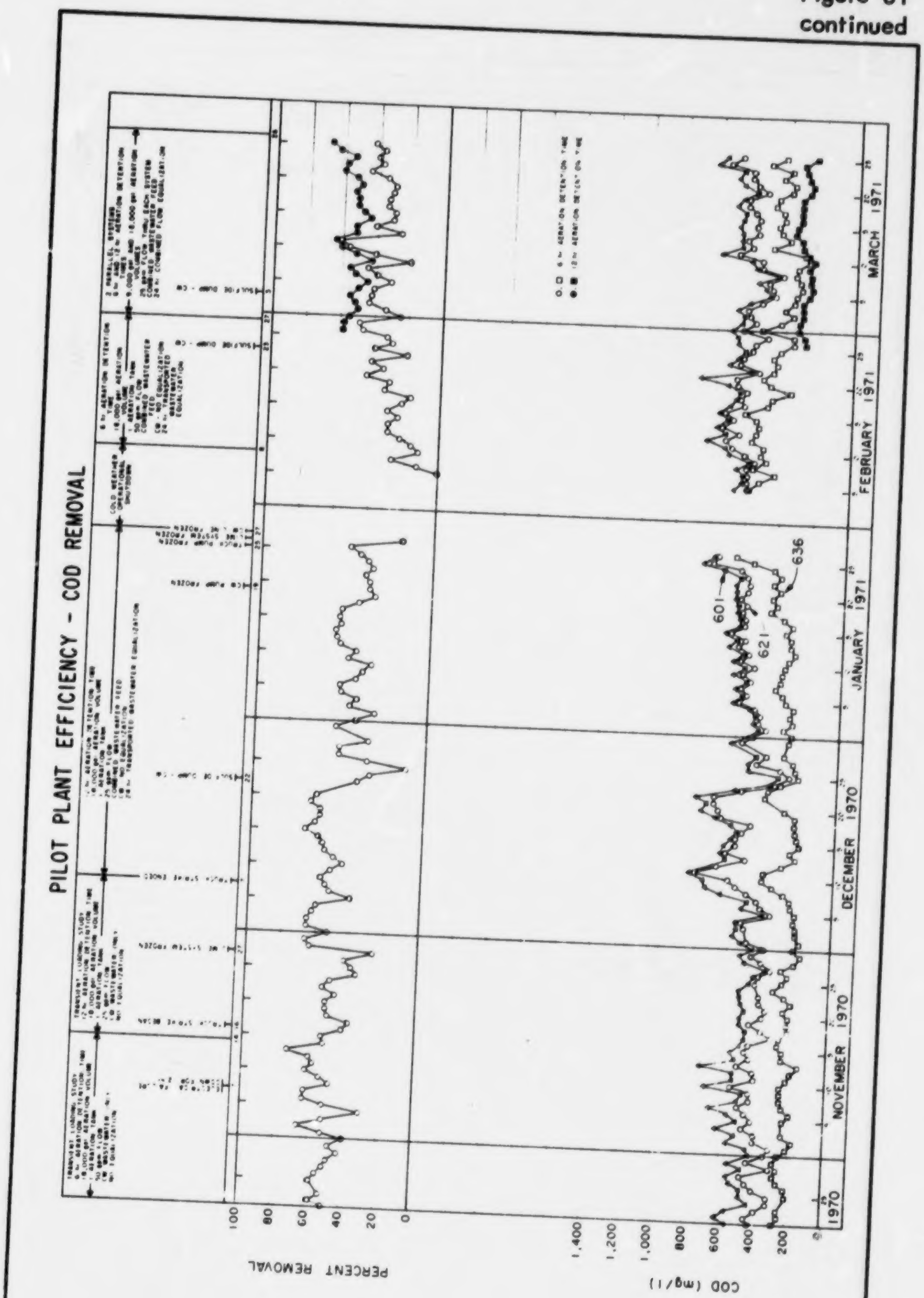


Figure 82

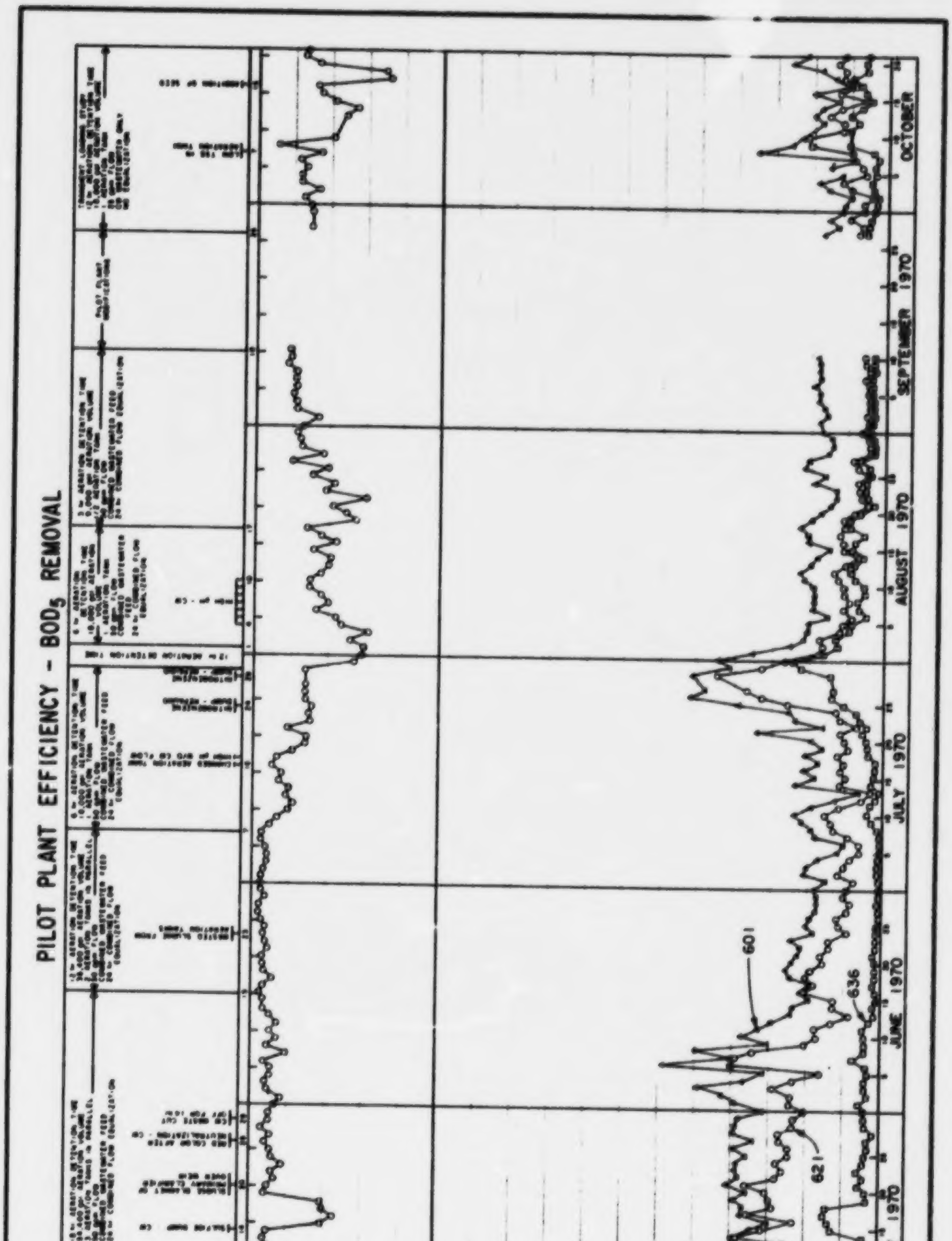
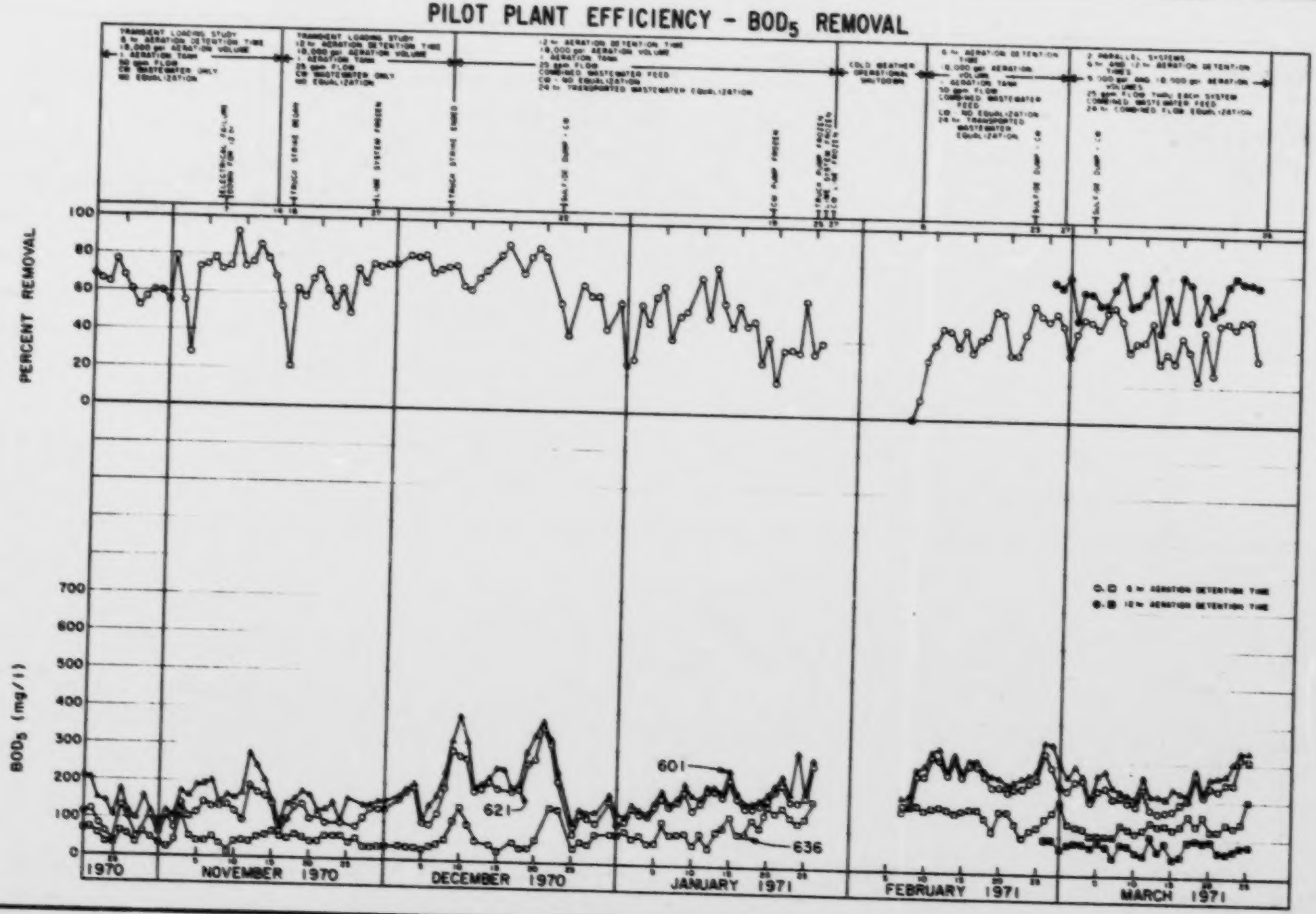
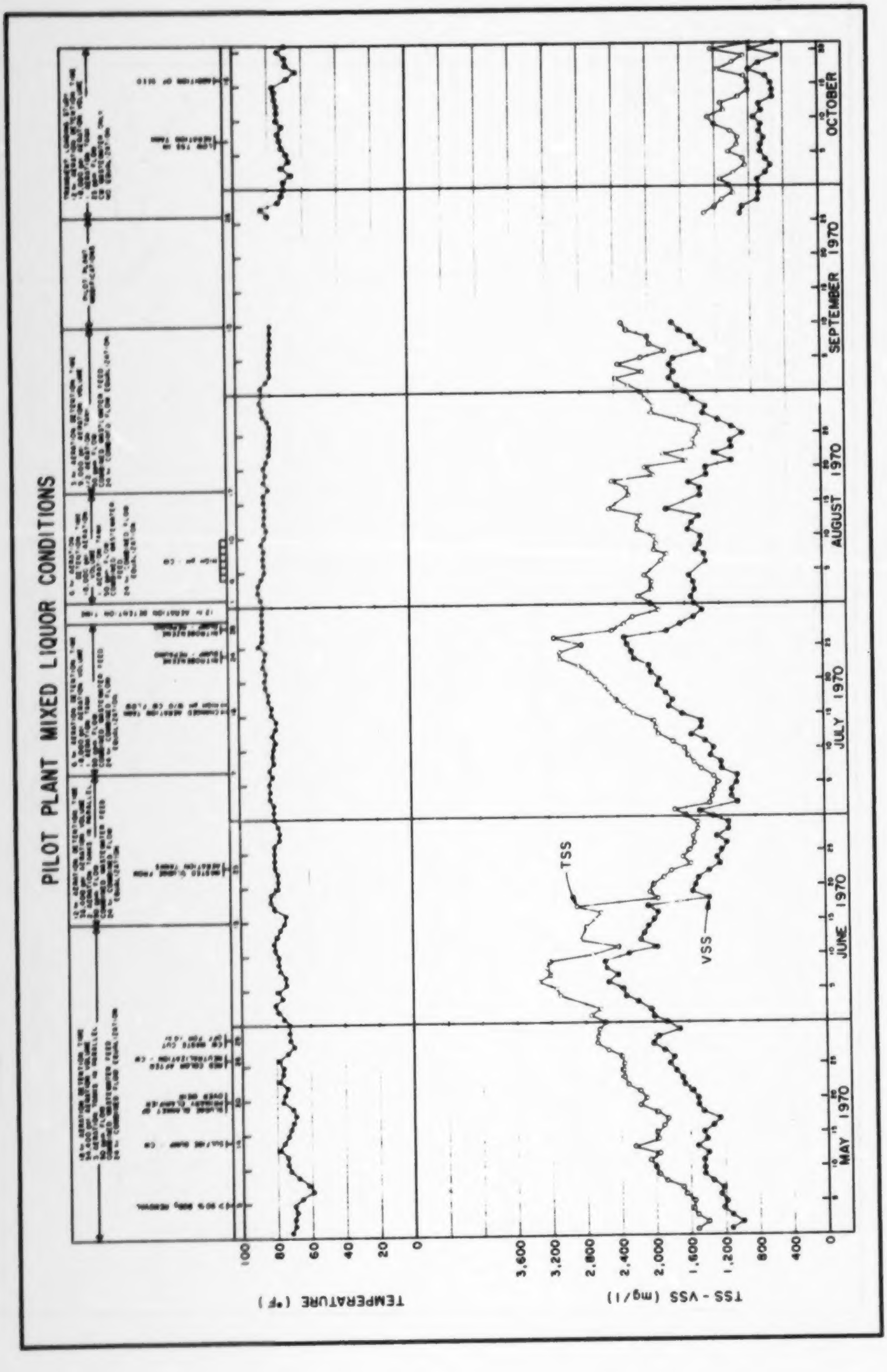

PILOT PLANT EFFICIENCY - BOD5 REMOVAL
PERCENT REMOVAL
100
80
60
40
20
0
BOD5 (mg/l)
700
600
500
400
300
200
100
0
601
636
621
MAY 1970
JUNE 1970
JULY 1970
AUGUST 1970
SEPTEMBER 1970
OCTOBER

Figure 82 (Continued)



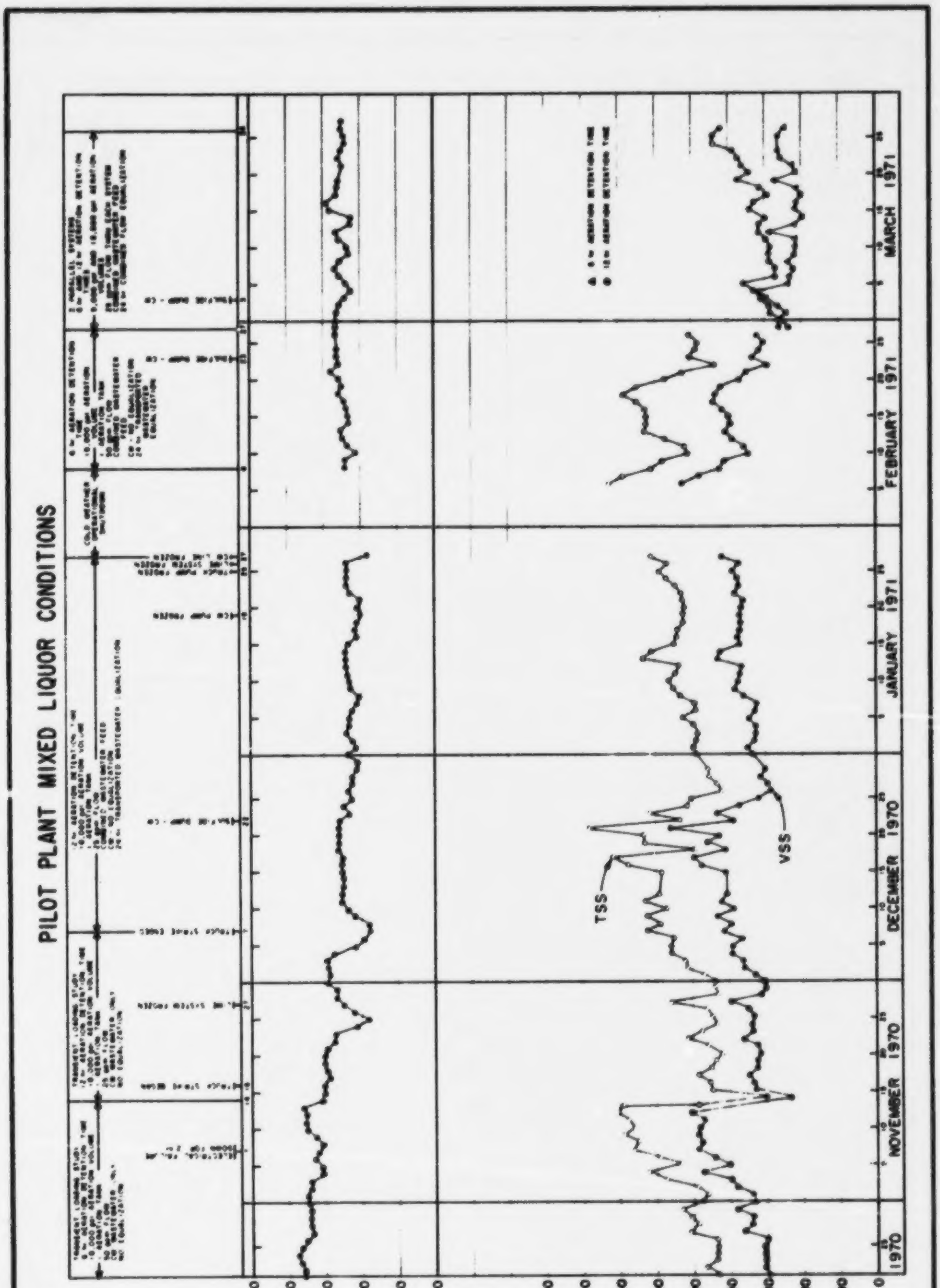
PILOT PLANT EFFICIENCY - BOD_5 REMOVAL

Figure 83



PILOT PLANT MIXED LIQUOR CONDITIONS

Figure 83 (cont.)



with a decrease in aeration detention time as shown in Figures 81 and 82. Moreover, the removal efficiencies dropped markedly during the winter loading series due to the cold weather effects on the biological system. Recognizing that as the organic loading increases, the removal efficiency decreases, data groups having an approximate BOD_5 loading of 0.2 lbs BOD_5/lb MLVSS/day were selected at each of the four detention periods tested during the summer months. Figure 84 presents this summarized data and reflects the removal efficiencies across the total pilot plant system. As noted, the removal efficiency was the highest at the twelve hour detention time and decreased as the detention time was decreased and the organic load was increased. The organic removal efficiencies across the aeration basin alone decrease even more dramatically as shown in Figure 85.

The removal efficiencies observed during the winter months were lower as compared to the summer operation. Since biological systems exhibit temperature dependence and since the effluent quality standards necessitate accurate prediction as to the winter removal efficiencies of a biological system, a complete analysis of the temperature effects was undertaken and is described in the following section.

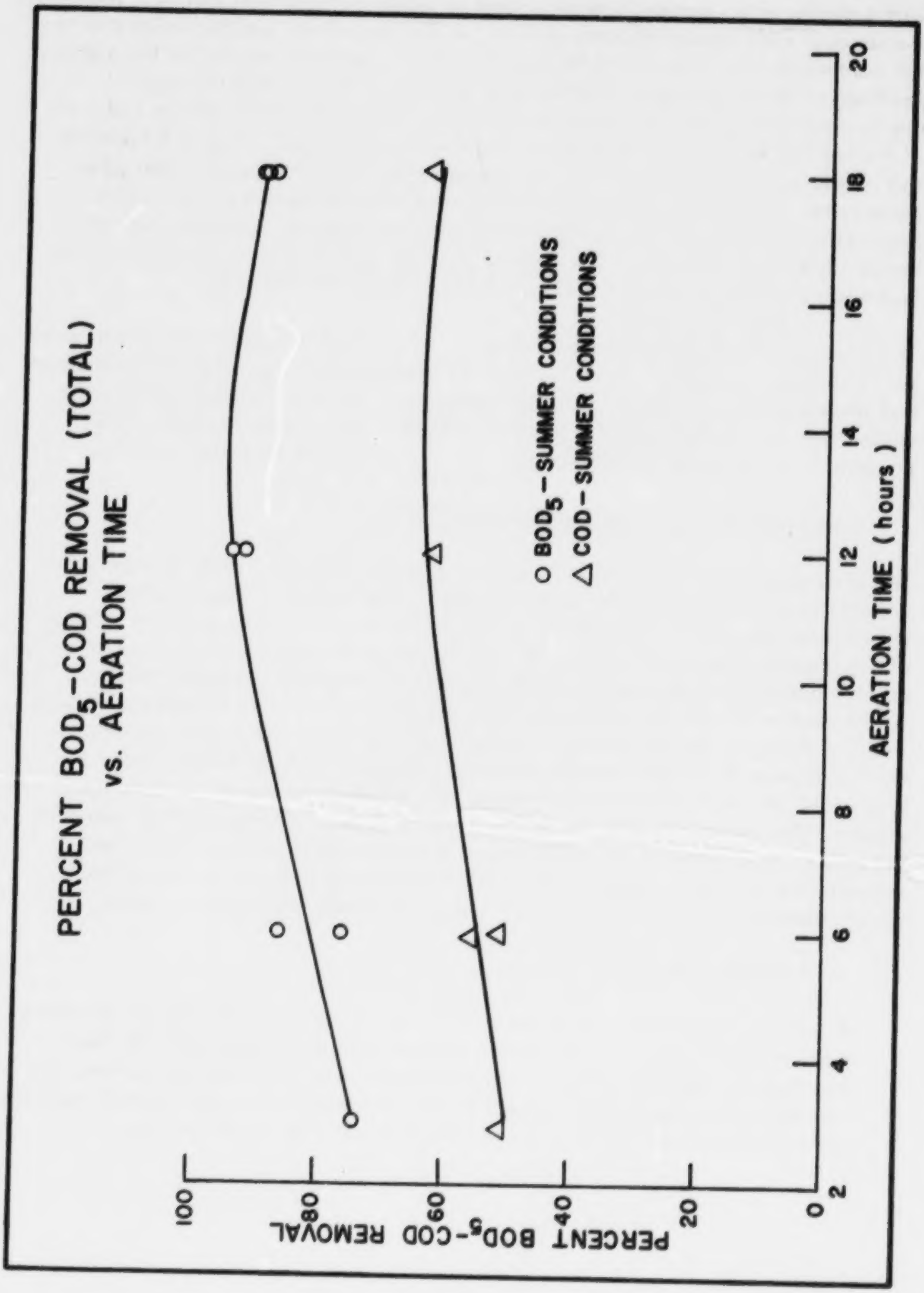
Cold Weather Effects on the Biological System

The expanding use of mechanical aerators for oxygenating activated sludge basins coupled with increasingly stringent temperature and organic effluent criteria underscores the need for accurately predicting temperature balances in the design of the regional treatment system. It should first be recognized that a mechanically aerated activated sludge basin is both a cooling pond and a biological reactor. As the degree of heat dissipation dictates the equilibrium basin temperature which in turn influences the efficiency of organic removal via biochemical oxidation, the importance of temperature prediction is apparent. Paradoxically, many biological treatment systems are designed with little or no reference to thermal effects. The purpose of this discussion, therefore, is to present a design approach for predicting a temperature profile across a mechanically aerated basin, and estimate the resultant biological removal capacity and effluent quality of the system based on the data accumulated during the winter pilot plant studies.

General Review:

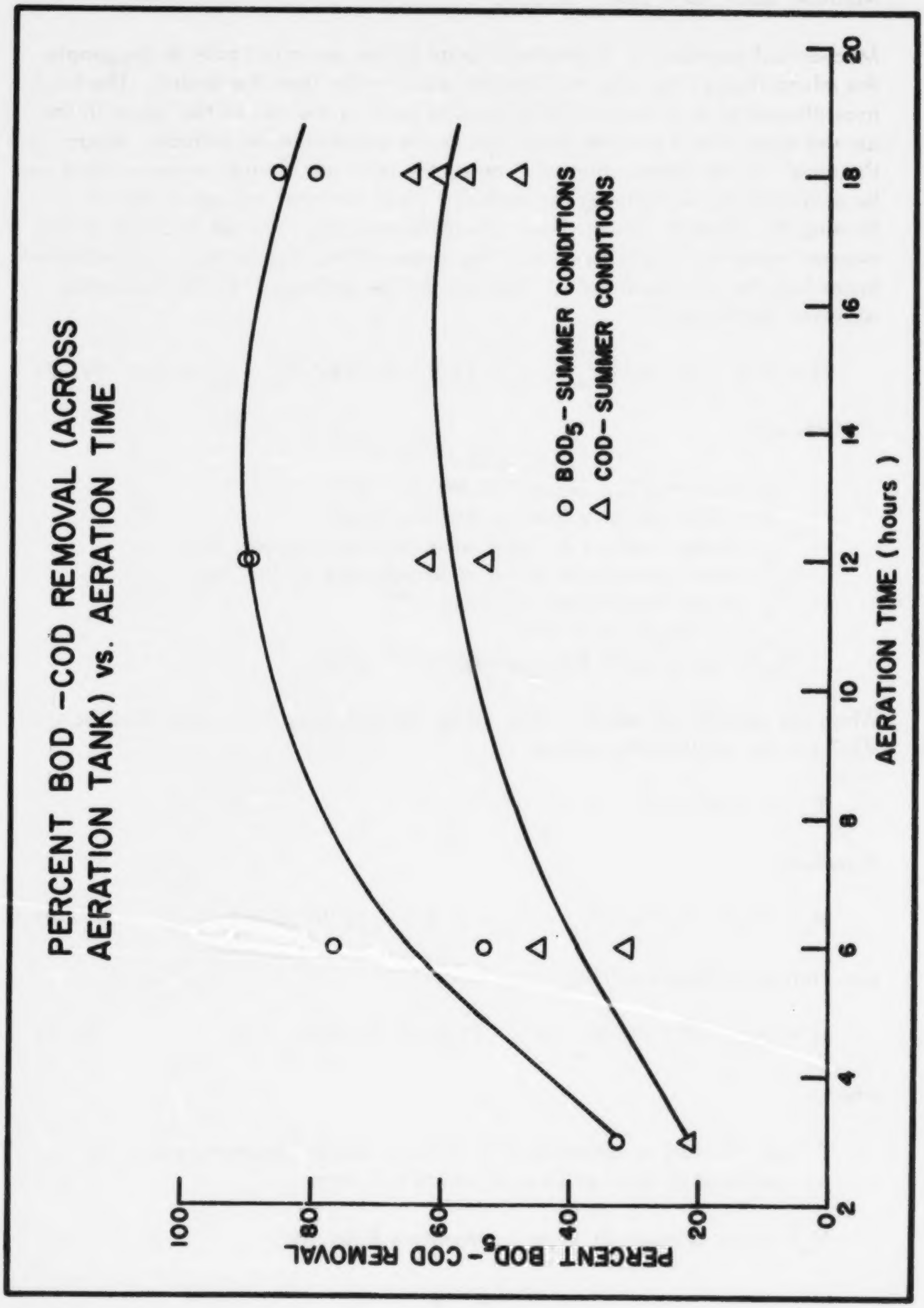
A review of pertinent historical information is necessary in order to provide a basis for developing a rational temperature-prediction approach. As heat loss from mechanical aerators, temperature effects on biological systems, and regulatory constraints with respect to effluent temperature and organic residuals are all interrelated, each of these aspects is included in this review.

Figure 84



PERCENT BOD_5-COD REMOVAL (TOTAL) vs. AERATION TIME

Figure 85


PERCENT BOD -COD REMOVAL (ACROSS
AERATION TANK) vs. AERATION TIME
○ BOD$_5$-SUMMER CONDITIONS
△ COD-SUMMER CONDITIONS
PERCENT BOD$_5$-COD REMOVAL
100
80
60
40
20
0
2
4
6
8
10
12
14
16
18
20
AERATION TIME (hours)

Aeration Basin Heat Loss

Mechanical aerators in activated sludge basins serve not only to oxygenate the mixed liquor, but also to increase heat transfer from the basins. The total heat dissipated in a mechanically aerated basin is the sum of the losses in the aerator spray cloud and the losses due to the exposed water surface. Assuming the usual "water warmer than air" case, the heat loss through a spray cloud can be estimated by multiplying the enthalpy (heat content) change of the air flowing the cloud by the air flow rate (Reference 4). The net heat loss at the exposed water surface is the sum of the evaporation, convection, and radiation losses less the solar heat gain. This loss can be estimated by the following equation (Reference 5).

$$H = 75\ (1 + 0.1 \cdot W)\ (V_w - V_a) + (1.8 + 0.12\ W)\ (T_w - T_a) - H_s \qquad \text{(VI-7)}$$

where:

H = net heat loss (BTU/ft^2 x hr)
W = wind velocity (mph) - tree top level
V_w = vapor pressure of water at temperature T_w (in. Hg)
V_a = vapor pressure of water at temperature T_a (in. Hg)
T_w = water temperature at surface (°F)
T_a = air temperature (°F)
H_s = average solar heat gain (BTU/ft^2 x hr)

When the equilibrium pond or river water temperature, E, is used, Equation VI-7 can be modified by setting:

$$T_w = E \text{ and } H = 0$$

therefore:

$$H_s = 75\ (1 + 0.1\ W)\ (V_e - V_a) + (1.8 + o.12\ W)\ (E - T_a) \qquad \text{(VI-8)}$$

substituting in Equation VI-7:

$$H = 75\ (1 + 0.1\ W)\ (V_w - V_e) + (1.8 + 0.12\ W)(T_w - E) \qquad \text{(VI-9)}$$

where:

E = equilibrium temperature (°F) -- i.e., water temperature of undisturbed pond or river at which H is zero

V_e = vapor pressure of water temperature E (in. Hg)

The total heat loss from a mechanically aerated basin can be predicted using this approach, although several assumptions are required. The validity or degree of accuracy can be established by comparing actual heat loss in existing systems to the calculated values.

Temperature Effects on Biological Systems

Temperature influences the rate of chemical and biochemical reactions. In the range of normal biological activity (5° to $35^\circ C$), the biochemical organic removal rate, K, approximately doubles for each $10^\circ C$ rise in temperature. According to the Van't Hoff-Arrhenius equation, K would vary with temperature as follows:

$$d \ln K/dt = E_a/RT^2 \qquad \text{(VI-10)}$$

where:

K = organic removal rate coefficient
T = absolute temperature
E_a = energy of activation constant
R = universal gas constant

The most traditional expression for relating the organic removal rate (via biochemical oxidation) with temperature is the Phelps equation (Reference 6):

$$K_T = K_{20^\circ C}\, \theta^{(T-20)} \qquad \text{(VI-11)}$$

where:

K_T = organic (BOD) removal rate coefficient at temperature T
$K_{20^\circ C}$ = organic (BOD) removal rate coefficient at $20^\circ C$
T = liquid temperature, °C
θ = temperature coefficient

The coefficient, θ, is a function of many variables; namely, the nature of the wastewater and the type of process. For example, Eckenfelder has reported θ values ranging from 1.06 to 1.09 for a temperature range of $10^\circ C$ to $30^\circ C$ (Reference 7). Wuhrmann has reported θ to be 1.0 for activated sludge, treating domestic wastes (Reference 8), and Howland has reported θ to be 1.035 for trickling filters (Reference 9). Based on the pilot plant studies, a θ approaching 1.05 was calculated. This indicates a more pronounced temperature effect when treating soluble industrial wastes as compared to treating domestic wastes of a colloidal and suspended nature. This is logical when considering that the organic removal via physical entrapment of colloidal and suspended BOD (biosorption) on the activated floc is less temperature dependent than the biochemical oxidation of soluble BOD. The results of the pilot plant study plus reported temperature effects on existing industrial waste activated sludge plants are shown in Figure 86. This temperature-efficiency relationship illustrates the importance of recognizing this effect when designing activated sludge systems, particularly for soluble industrial wastes, and predicting the effluent quality during the most critical winter months.

Technical Approach and Justification:

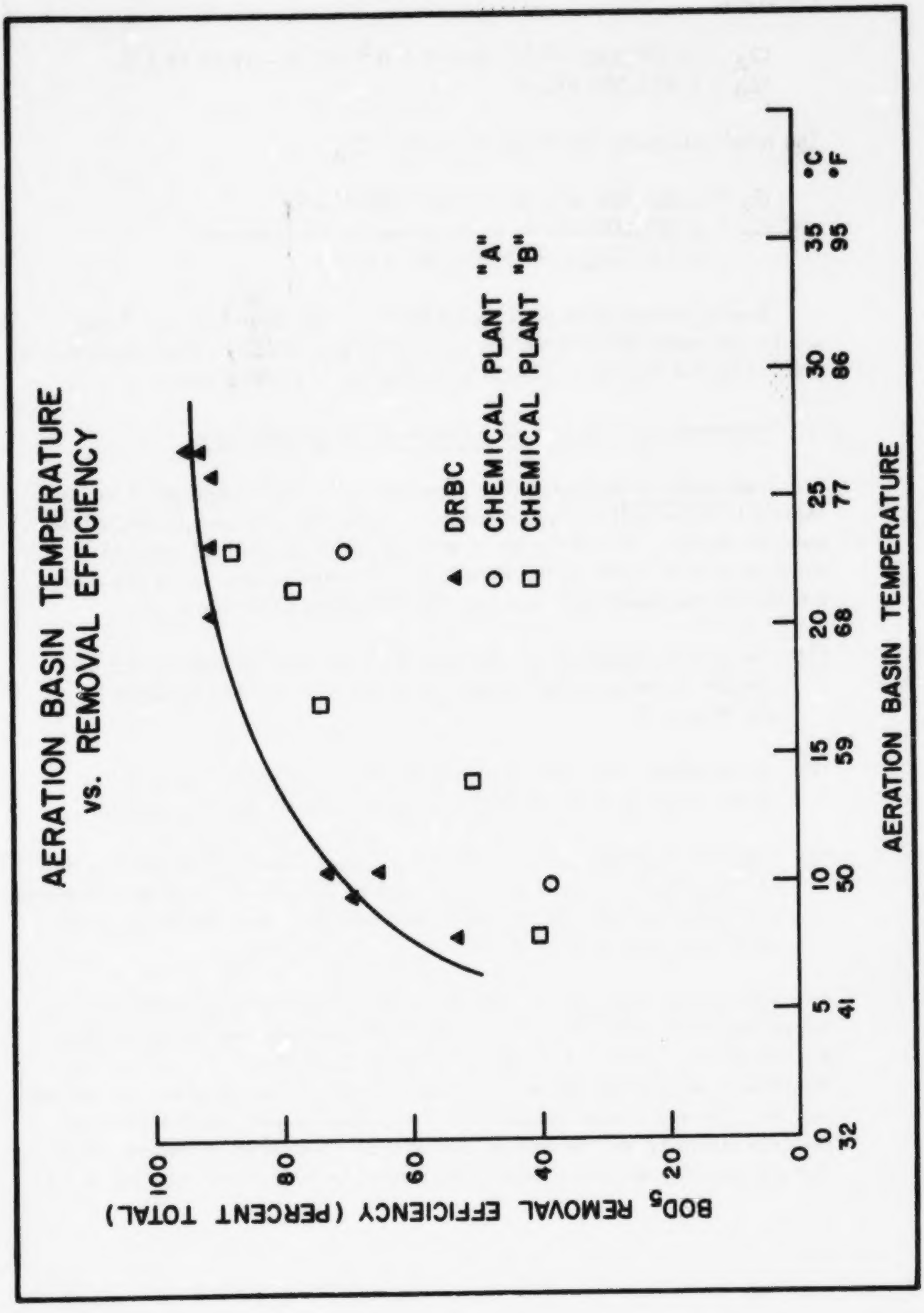
The approach for predicting temperatures in mechanically aerated basins as described herein includes the calculation of heat loss attributable to the aeration spray and the predicted loss through the surface. The spray heat loss is calculated from the differential enthalpy of the air flow through the spray cloud. The cross-sectional area of the spray pattern exposed to the air flow from the design mechanical aeration unit must be known, the velocity of air through the spray estimated, and the approach and exit air temperature predicted. The surface losses can be estimated by Equation VI- 7 or VI-9 which require climatological data for the area in question. This includes the selection of design values for relative humidity, wind velocity, ambient air temperature, solar radiation, equilibrium water temperature (if applicable), and the influent liquid temperature. Once these two heat loss components are estimated, the total heat loss can be used to predict the aeration basin temperature as a function of the influent water temperature. The biological response in terms of organic removal then can be correspondingly calculated.

In order to establish a valid basis for this procedure, four existing aeration basins using mechanical aerators were surveyed. Two basins were in Texas and two were in Illinois. Climatological data, influent and basin temperatures, and mechanical aerator spray patterns were obtained for each basin. The calculated heat loss using the aforementioned procedure was then compared to actual heat loss to demonstrate the degree of accuracy. Example calculations for one of the four basins is presented as follows:

Survey Information -

Basin Location	Southern Illinois
Wastewater Flow	1,300 gpm
Wastewater Temperature In	98°F
Wind Velocity (tree top level)	8.1 mph
Aeration Basin Temperature	89°F
Ambient Air Temperature	89°F
Equilibrium Temperature (based on river temperature)	82°F

Figure 86


AERATION BASIN TEMPERATURE
vs. REMOVAL EFFICIENCY
▲ DRBC
○ CHEMICAL PLANT "A"
□ CHEMICAL PLANT "B"
BOD5 REMOVAL EFFICIENCY (PERCENT TOTAL)
100
80
60
40
20
0
0 32
5 41
10 50
15 59
20 68
25 77
30 86
35 95
°C °F
AERATION BASIN TEMPERATURE

therefore:

$$Q_A = (6{,}770{,}000\ ft^3/hr)\ (lb/14.2\ ft^3)\ (47.0 - 39.0)\ BTU/lb$$
$$Q_A = 3{,}820{,}000\ BTU/hr$$

The total calculated heat loss $Q_c = Q_s + Q_A$

$$Q_c = 2{,}360{,}000\ BTU/hr + 3{,}820{,}000\ BTU/hr$$
$Q_c = 6{,}180{,}000$ BTU/hr as compared to the observed value, $Q_{Act.}$ of 5,850,000 BTU/hr

Similar comparisons were made for the other three basins. These results are reported in Table 35, indicating the validity of this approach in predicting the heat loss through a mechanically aerated basin.

Temperature Calculations - Deepwater Regional Plant:

Information Furnished - The proposed activated sludge plant for the regional system will include mechanical aeration with completely mixed aeration basins. The first step in estimating the biological removal efficiency is to estimate the aeration basin temperature using the aforementioned procedure and based on the following conditions:

(1) The climatological data obtained from the weather bureau station closest to the proposed construction site as tabulated in Table 36 and Figure 87.

(2) The temperature of the wastewater into the aeration basin being in the range of 45°F to 65°F during the coldest day of operation.

(3) Parallel aeration basins will be used. Each basin will receive a flow of 12.0 MGD, occupy a surface area of 75,250 ft^2, and be oxygenated and mixed by ten 100 HP slow speed aerators, each having a cross-sectional spray area of 80 ft^2.

Information Required - The relationship between the aeration basin temperature and the influent wastewater temperature must be developed for the coldest month. The design ambient temperature is taken as the 10 percent probability value of the daily mean temperature for the coldest month. The mean wind velocity for the coldest month and the average relative humidity for the coldest day will be considered as design values. The air velocity at the surface is assumed to be half of the tree top value.

Dew Point	70°F
Basin Dimensions	375' x 120'

Mechanical Aerators, five 20 HP fixed mounted, slow speed with $63.3\ ft^2$ cross-sectional spray area per unit;

Total cross-sectional spray area = 5(63.3) = 316.5 ft^2

Actual Heat Loss -

Actual Heat Loss, $Q_{Act.} = (1,300\ gpm)\ (8.34\ lb/gal)\ (60\ min/hr)\ (98^\circ - 89^\circ)\left(\frac{BTU}{lb - {}^\circ F}\right)$

$Q_{Act.} = 5,850,000\ BTU/hr$

Calculated Heat Loss -

A. Heat Loss from non-aerated surface:

Using Equation VI-9 where V_w @ 89°F = 1.375 in. Hg

V_e @ 82°F = 1.106 in. Hg

$H = 75\ [1 + 0.1(8.1)]\ [1.375 - 1.106] + [1.8 + 0.12(8.1)\ (89-82)]$

$H = 75\ (1.81)\ (.269) + (2.77)\ (7)$

$H = 56.2\ BTU/ft^2/hr$

The unaerated area of the basin = 42,000 ft^2 (assuming spray diameter = 28 ft)

The heat loss from the non-aerated surface, Q_s, is therefore:

$Q_s = \left(\frac{56.2\ BTU}{ft^2 \times hr}\right)(42,000\ ft^2) = 2,360,000\ BTU/hr$

B. Heat Loss from the five mechanical aerators, Q_A, is:

$Q_A =$ Air flow (h_a in - h_a out) (VI-12)

where:

h_a in = enthalpy of air into the spray, BTU/lb

h_a out = enthalpy of air out of spray, BTU/lb

air flow = (air velocity at surface of water) (total cross-sectional area of spray)

Assume air velocity at surface equals 50 percent of the air velocity at tree level.

air flow = (4.05 mph) (5,280 ft/mi) (316.5 ft^2) = 6,770,000 ft^3/hr

h_a in = 39.0 BTU/lb

h_a out = 47.0 BTU/lb at est. 85°F and 90 percent relative humidity

(Conditions for air leaving spray cloud based on spray pond design given in Perry's Chemical Engineering Handbook, Reference 10.)

TABLE 35

AERATION BASIN HEAT LOSS COMPARISON

	Actual Heat Loss, $Q_{Act.}$ BTU/hr	Calculated Heat Loss, Q_c BTU/hr
Aeration Basin No. 1 (Illinois)	5,850,000	6,180,000
Aeration Basin No. 2 (Illinois)	14,300,000	11,630,000
Aeration Basin No. 3 (Texas)	9,120,000	10,150,000
Aeration Basin No. 4 (Texas)	11,000,000	13,430,000

TABLE 36

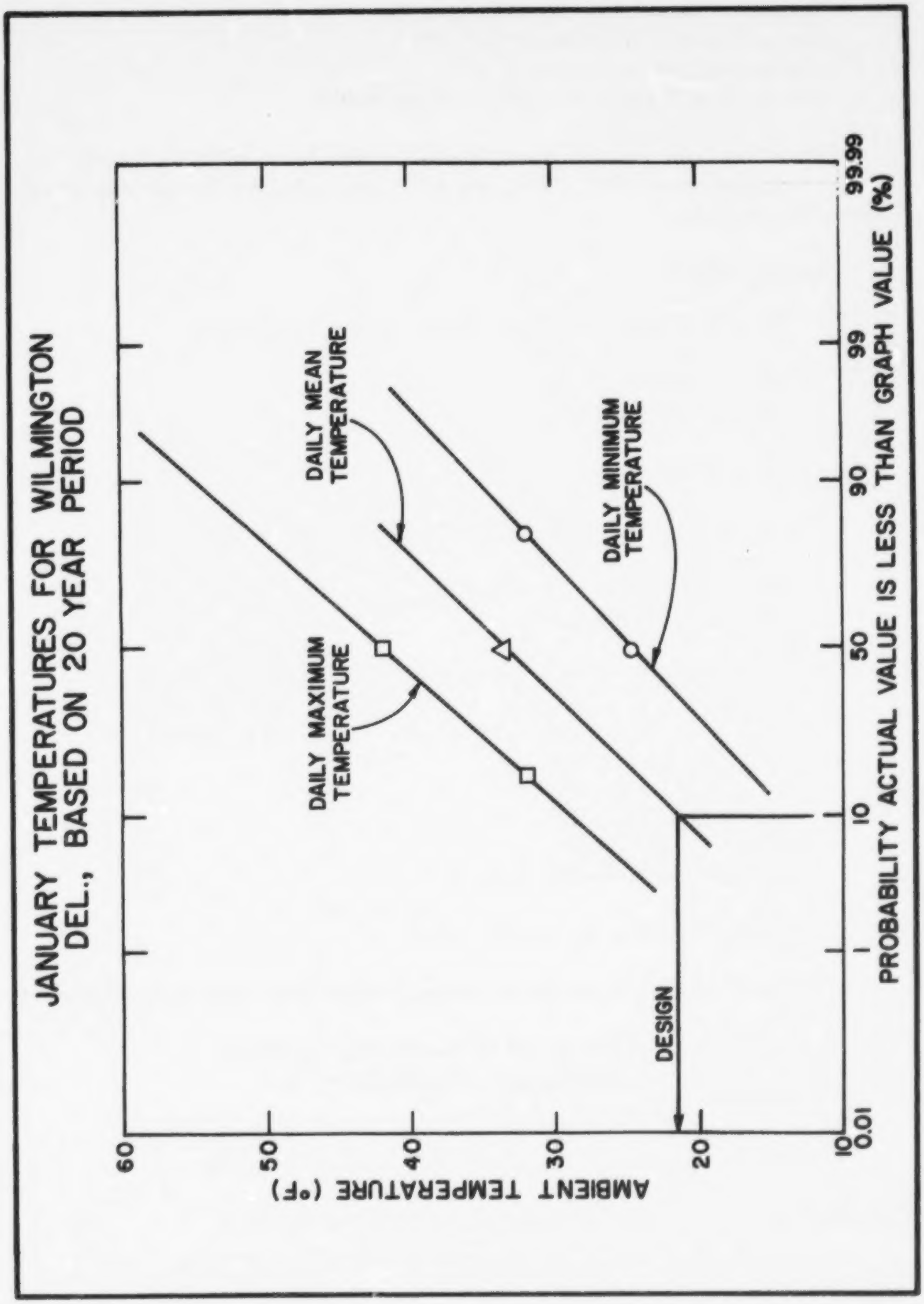
CLIMATOLOGICAL DATA FOR PROPOSED TREATMENT SITE*

	December, 1970		January, 1971		February, 1971	
Time of Day	Ambient Temp. °F	Dew Point °F	Ambient Temp. °F	Dew Point °F	Ambient Temp. °F	Dew Point °F
0100	34	26	26	17	32	26
0400	33	25	25	17	31	25
0700	32	25	24	17	31	25
1000	37	25	29	18	36	25
1300	41	25	33	17	41	25
1600	40	26	33	16	41	26
1900	37	27	28	17	36	27
2200	35	27	26	16	34	27
AVERAGE	36.2°F		27.8°F		36.3°F	

* Climatological data from Wilmington, Delaware airport

A. January is the coldest month and considered for design.
B. The probability of the ambient air temperature (daily maximum, mean, and minimum) being equal to or less than the graph value for the month of January is shown in Figure 87.
C. The average relative humidity for the coldest day in January is 71.2 percent.
D. The mean wind velocity for January based on a ten year average is 9.0 mph (tree-top level).
E. Solar heat gain = 24 BTU/hr/ft^2

Figure 87



JANUARY TEMPERATURES FOR WILMINGTON DEL., BASED ON 20 YEAR PERIOD

Solution - From Table 36 and Figure 87:
Design Ambient Temperature = 21.5°F
Design Relative Humidity = 71.2%
Design Wind Speed = 9.0 mph (tree top level)*

Heat Loss From Exposed Water Surface -- Use Equation VI-7; assume basin temperatures of 40°F, 50°F, and 60°F, and calculate the corresponding inlet temperatures.

For $T_w = 40°F$;

$$H = 75\ (1 + 0.1)9.0(.246 - .127)\ + [1.8 + 0.12\ (9.0)]\ (40 - 21.5)\ -24$$
$$H = 17.0 + 53.3 - 24$$
$$H = 46\ BTU/hr \times ft^2$$

For $T_W = 50°F$;

$$H = 75\ [1 + 0.1(9.0)]\ [.362 - .127]\ + [1.8 + 0.12(9.0)]\ [50 - 21.5]\ - 24$$
$$H = 33.5 + 82.0 - 24$$
$$H = 91.5\ BTU/hr \times ft^2$$

For $T_w = 60°F$;

$$H = 75\ [1 + 0.1(9.0)]\ [.520 - .127]\ + [1.8 + 0.12(9.0)]\ [60 - 21.5]\ -24$$
$$H = 56.0 + 111 - 24$$
$$H = 143\ BTU/hr \times ft^2$$

Heat Loss Due to Aerator Spray --

$$Q_A = \text{Air Flow}\ (h_a\ \text{in} - h_a\ \text{out})$$

Air flow through 10 aerators assuming surface wind velocity at 50 percent:

$$\text{Air Flow} = (4.5\ \text{mph})(80\ ft^2/\text{aerator})\ (10\ \text{aerators})\ (5{,}280\ ft/mi) = 19{,}000{,}000\ ft^3/hr$$

---

*Constrants in Equation VI-7 assume wind speed at tree top level.

For air @ 21.5°F and 71.2% relative humidity:

h_a in = 6.9 BTU/lb

To obtain enthalpy of air leaving spray cloud, the approach to saturation is estimated at a temperature of 7°F with 90 percent saturation:

	40°F	50°F	60°F
T_a out	33°F	43°F	53°F
h_a out	11.8 BTU/lb	16.1 BTU/lb	21.5 BTU/lb
air	12.4 ft^3/lb	12.7 ft^3/lb	13.0 ft^3/lb

Applying Equation VI-12;

$$Q_{A(40°F)} = (19{,}000{,}000\ ft^3/hr)\ (lb/12.4\ ft^3)(11.8 - 6.9)BTU/lb$$
$$= 7{,}500{,}000\ BTU/hr$$

$$Q_{A(50°F)} = (19{,}000{,}000\ ft^3)(lb/12.7\ ft^3)(16.1 - 6.9)BTU/lb$$
$$= 13{,}700{,}000\ BTU/hr$$

$$Q_{A(60°F)} = (19{,}000{,}000\ ft^3/hr)(lb/13.0\ ft^3)(21.5 - 6.9)\ BTU/lb$$
$$21{,}300{,}000\ BTU/hr$$

The Q_s values are calculated as follows:

The unaerated area of the basin assuming a spray diameter of 35 feet is:

Area = 75,250 ft - 10 (.785)$(35)^2$
Area = 65,650 ft^2 per basin

$$@\ 40°F, 46\left(\frac{BTU}{ft^2 \times hr}\right)(65{,}650\ ft^2) = 3{,}020{,}000\ BTU/hr$$

$$@\ 50°F, 91.5\left(\frac{BTU}{ft^2 \times hr}\right)(65{,}650\ ft^2) = 6{,}007{,}000\ BTU/hr$$

$$@\ 60°F, 143\left(\frac{BTU}{ft^2 \times hr}\right)(65{,}650\ ft^2) = 9{,}388{,}000\ BTU/hr$$

Calculation Summary --

Assumed Basin Temperature	40°F	50°F	60°F
Q_s, BTU/hr	3,020,000	6,007,000	9,388,000
Q_A, BTU/hr	7,500,000	13,700,000	21,300,000
Total Q	10,520,000	19,707,000	30,688,000

At a hydraulic flow of 12.0 MGD;
(12,000,000 gpd)(8.34 lb/gal)(day/24 hrs) = 4,170,000 lb/hr
T = Q/flow

$T_{40^\circ F}$ = 10,520,000/4,170,000 = 2.5°F
$T_{50^\circ F}$ = 19,707,000/4,170,000 = 4.7°F
$T_{60^\circ F}$ = 30,688,000/4,170,000 = 7.4°F

Inlet Temperatures = 42.5°F, 54.7°F, and 67.4°F respectively.

The relationship between the aeration basin temperature and the influent wastewater temperature for the coldest month is shown in Figure 88. Based on the observations at the pilot plant during January, 1971, the average influent temperature to the aeration basins was 52°F during days when the average ambient temperature was 20°F to 22°F. Assuming the 52°F entrance temperature, the aeration basin temperature would be 48°F as shown in Figure 88. The minimum predicted removal efficiency at this temperature, in terms of BOD, is then 66 percent based on the pilot plant studies as shown in Figure 86.

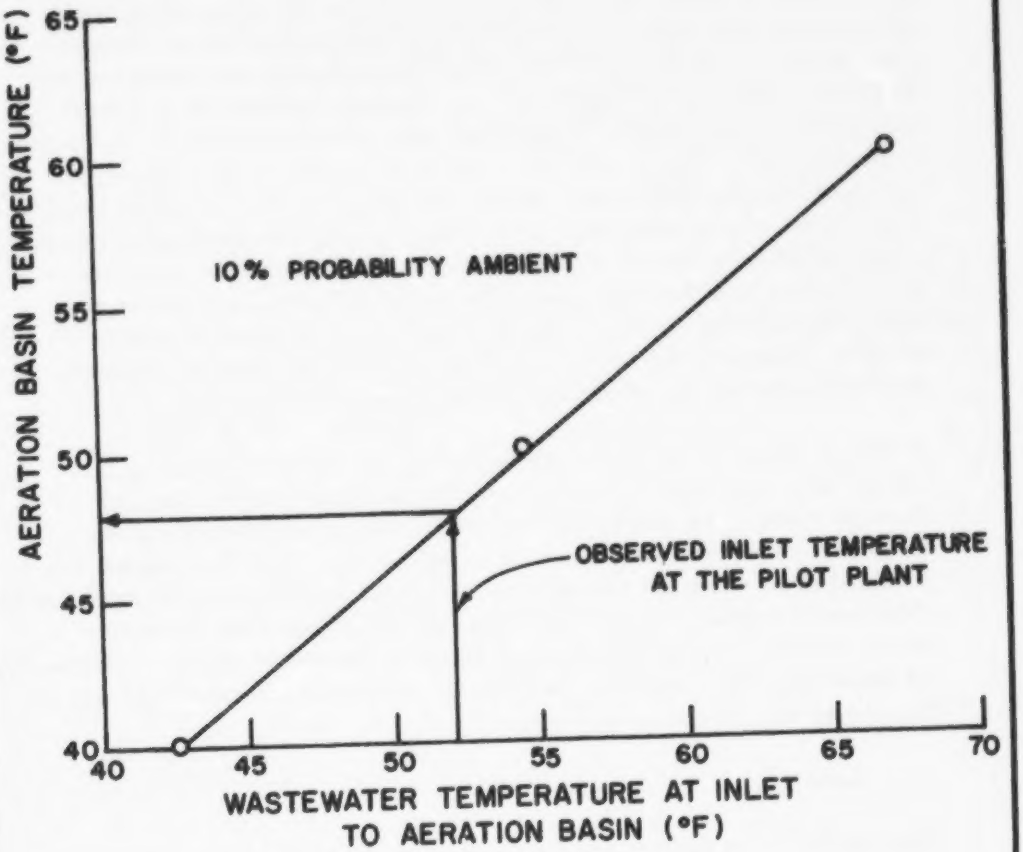
It should be recognized that the observed inlet temperature during the pilot plant studies may be lower than that of an interceptor flow because of the physical characteristics of the pilot system. Heat losses occurred during wastewater storage and equalization prior to the aeration system. Since the proposed regional system will not include storage or equalization, a slightly higher inlet temperature could be expected even considering losses in the participant equalization basins and in transmission.

Summary:

In summary, it is obvious that heat loss calculations need to be considered when formulating the conceptual design for the activated sludge process treating industrial wastewaters. This is particularly true when specific effluent criteria must be observed throughout the year. The Delaware River Basin Commission, in its standards for the Delaware estuary, has limited the secondary treatment plant efficiency to a two-thirds override of effluent BOD during cold weather months where the operating temperature falls below 59°F. Based on this extra allowance, a removal efficiency of approximately 80 percent must be obtained during the winter months to conform with these standards. As the predicted maximum removal efficiency is 66 percent during the most severe winter conditions, the biological system alone would not provide the necessary treatment during this time. Effluent polishing using activated carbon columns, however, will satisfy this particular effluent criteria throughout the year.

Figure 88

AERATION BASIN TEMPERATURE vs. BASIN INLET TEMPERATURE FOR JANUARY CONDITIONS DEEPWATER REGIONAL TREATMENT PLANT



Transient Loading Effects on the Biological System

Biological systems, in addition to being temperature dependent, are also responsive to extreme variations in the organic load applied to the system. Equalization, therefore, was considered for dampening organic and flow variations prior to biological treatment. Several aspects of equalization are discussed in Section V. As most industrial wastewaters have varying organic characteristics resulting from batch type process operations, chemical spills, etc., the potential need for equalization at the regional plant site was investigated. Transient loading studies were therefore conducted at the pilot plant to determine the applicability of equalization.

Procedure:

The design of the pilot plant incorporated storage and equalization as a pretreatment process. A 71,000 gallon tank was provided which would allow a maximum equalization period of 23 hours at a flow of 50 gpm. The wastewater utilized for the summer loading series was equalized for 23 hours. The data obtained from these special tests served as a basis for comparing the effects of equalized and non-equalized flow on the biological system. Prior to the initiation of the winter loading series, transient loading studies were conducted using only the Chambers Works wastewater. This wastewater, when neutralized, exhibited many similar characteristics to that of the combined flow. During these tests, the equalization tank was bypassed. The analytical and sampling program remained the same as previously described. The 24-hour composite samples, however, did not reflect the instantaneous organic variations that were applied to the biological system.

A second series of transient loading studies was completed during the winter testing program. The flow regime during this series was such that the transported wastewater was equalized for 24 hours while the Chambers Works wastewater had no equalization. This flow regime was established to represent the regional treatment facility without equalization. Confirmatory tests, with 24-hour equalization of the total waste flow, were completed during the terminal phase of the winter studies. The results of these tests then served as the basis for comparing the equalized and non-equalized data of the winter tests.

Results:

The results of the summer and winter transient loading studies and the equalized comparative data are presented in Table 37. With respect to the summer conditions, the BOD and COD removal efficiencies are almost

TABLE 37

TRANSIENT LOADING EFFECTS ON THE BIOLOGICAL SYSTEM

FLOW REGIME	OPERATING TEMP. (°F)	ORGANIC LOADING (lb BOD/lb MLVSS/day)	PERCENT BOD$_5$ REMOVAL AERATION PROCESS	PERCENT BOD$_5$ REMOVAL TOTAL	PERCENT COD REMOVAL AERATION PROCESS	PERCENT COD REMOVAL TOTAL
SUMMER CONDITIONS						
All wastewater equalized for 24 hours - 6 hour aeration detention time	78	0.23	54.0	76.0	32.0	52.0
Chambers Works as total feed-no equalization - 6 hour aeration detention time	69	0.27	60.0	74.0	42.0	54.0
WINTER CONDITIONS						
All wastewater equalized for 24 hours - 6 hour aeration detention time	51.0	0.82	39.0	46.0	31.3	38.0
All wastewater equalized for 24 hours - 12 hour aeration detention time.	51.0	0.25	65.0	67.0	48.0	52.0
Chambers Works wastewater not equalized. All other wastewaters equalized for 24 hours - 6 hour aeration detention time	49.5	0.60	41.2	47.0	21.6	30.7
Chambers Works wastewater not equalized. All other wastewaters equalized for 24 hours - 12 hour aeration detention time	46.0	0.19	53.7	55.0	41.7	45.0
Chambers Works wastewater not equalized. All other wastewaters equalized for 24 hours - 12 hour detention time.	51.0	0.23	72.3	74.0	55.2	57.0

identical at the six-hour aeration detention time. Correspondingly, the removal efficiencies at six hours during the winter conditions are very similar. The data presented for the 12-hour detention time tests exhibited some difference which is primarily attributable to the difference in the operating temperature of the aeration basin. Based on this information, the variation in the organic characteristics of the combined wastewater as experienced at the pilot plant indicated little or no effect on the performance of the biological system. It should also be recognized that the variations of flow and wastewater constituents inherent with process operations will be dampened in the pre-equalization basins of the participants as well as in the conveyance system.

Summary:

In summary, there is no recommended need for equalization facilities at the regional treatment plant. Moreover, the proposed treatment facility will include completely mixed aeration chambers operated in parallel, providing additional operational flexibility and performance reliability.

Biological System Design Parameters and Coefficients

The pilot scale evaluation program was established not only to predict the reliability of the biological process, but also to develop the necessary design parameters based on the performance requirements of the proposed treatment system. The mathematical models which represent biological systems are presented in Section V of this report and serve as the basis for the following development of the biological design parameters.

Application of the STATPK Computer Program

As previously mentioned, computer techniques were utilized in the development of the biological design criteria. The basic approach in implementing the STATPK program was to select grouped biological data based on the modes of operation and environmental conditions, key punch this information on computer cards, and translate the results into design criteria. Two separate computer runs were made with the data groups delineated according to organic loadings and temperature conditions. Upon retrieval of the computer output, a complete review of the information was made based on the stated statistical significance of the data and the estimation of steady-state conditions. The design criteria and coefficients were then established and used for sizing the unit processes and predicting process performance.

Biological Design Coefficients

The biological design coefficients related to substrate removal rates, sludge production and oxygen requirements as determined from the STATPK program are presented graphically in Figures 89 through 94 and are summarized in Table 38.

TABLE 38

BIOLOGICAL DESIGN COEFFICIENTS

COEFFICIENT	BOD BASIS	COD BASIS
K - substrate removal rate (day^{-1}) (summer conditions)	0.0115	0.00485
K - substrate removal rate (day $^{-1}$) (winter conditions)	0.00487	0.00367
a - lbs sludge produced/lb BOD-COD removed	0.445	0.44
b - lbs sludge oxidized/lbs sludge/day	0.10	0.10
a' - lbs oxygen required/lb BOD-COD removed	0.913	0.699
b' - lbs oxygen required/lb sludge oxidized/day	0.0743	0.019

By incorporating these design criteria into the mathematical models as presented in Section V, the aeration detention time, oxygen requirements, and sludge production can be predicted as follows:

Conceptual Design Calculations

Aeration Detention Time:

(Summer Conditions - BOD basis)

Design basis:

1. BOD of raw wastewater = 230 mg/l (50 percentile value)
2. BOD removal in primary clarifier = 20%
3. Total removal = 87.5%
4. MLVSS = 1,500 mg/l

Figure 89

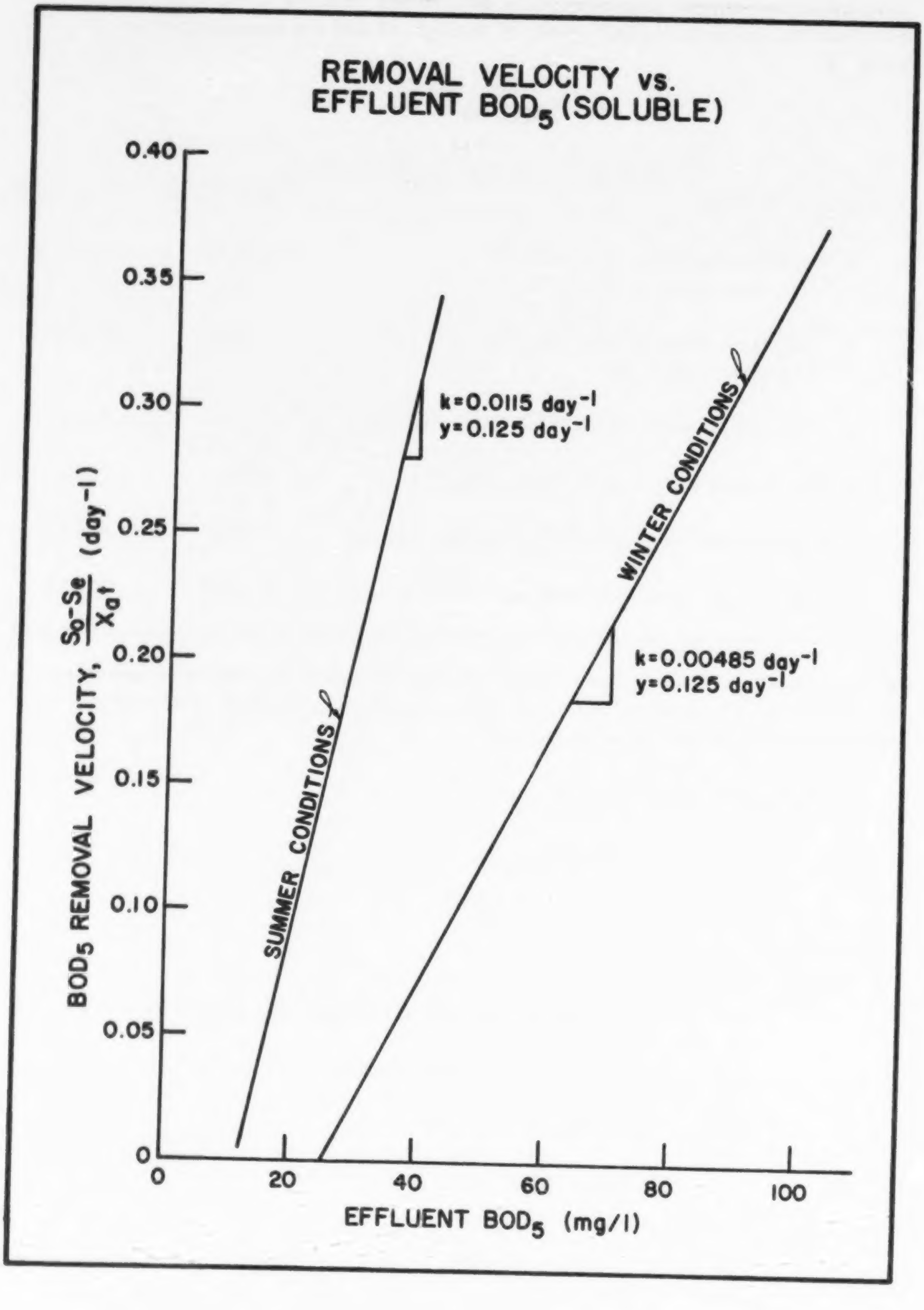
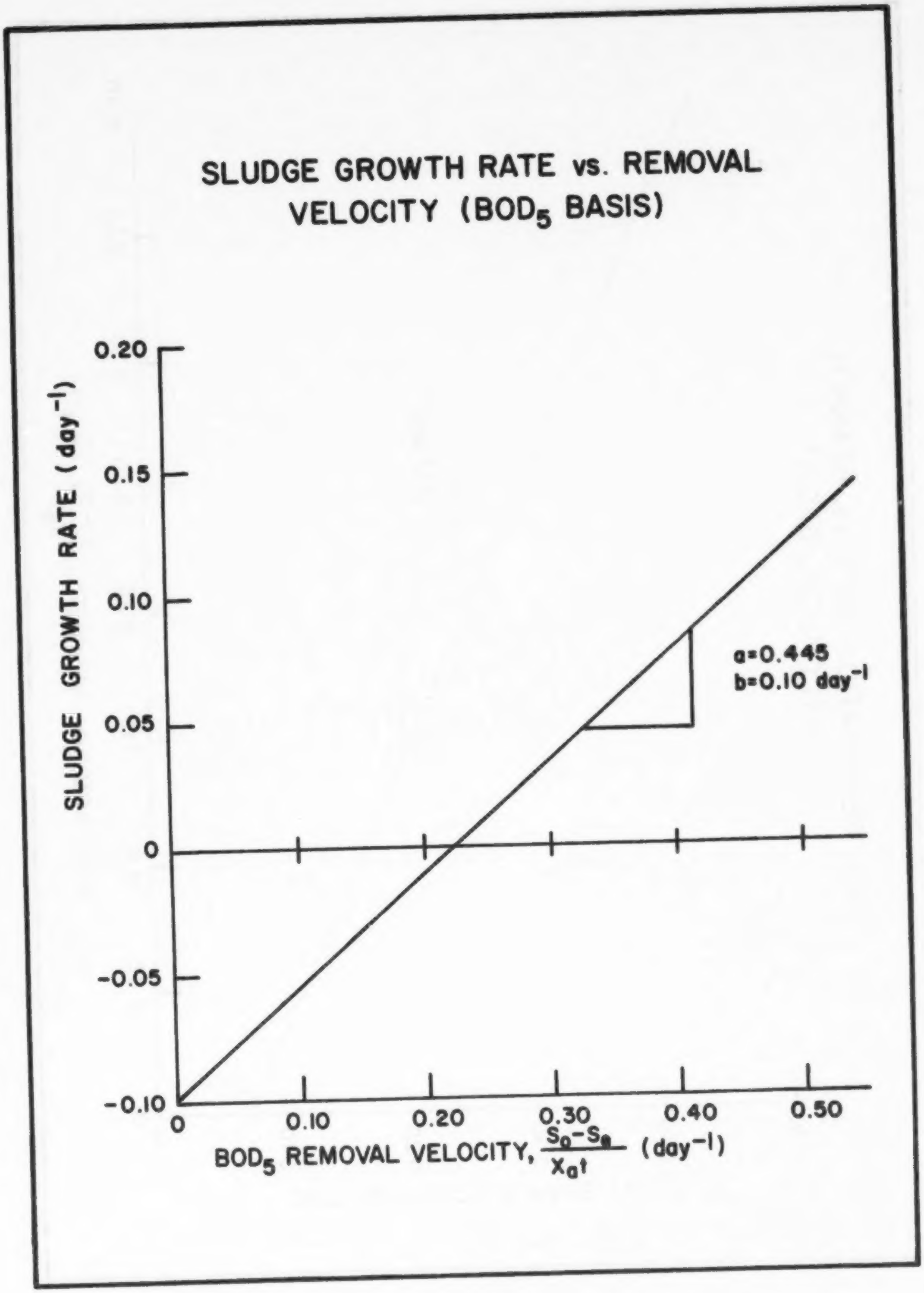

REMOVAL VELOCITY vs.
EFFLUENT BOD_5 (SOLUBLE)
BOD_5 REMOVAL VELOCITY, $\frac{S_o - S_e}{X_a t}$ (day^{-1})
0.40
0.35
0.30
0.25
0.20
0.15
0.10
0.05
0
SUMMER CONDITIONS
$k = 0.0115$ day^{-1}
$y = 0.125$ day^{-1}
WINTER CONDITIONS
$k = 0.00485$ day^{-1}
$y = 0.125$ day^{-1}
0
20
40
60
80
100
EFFLUENT BOD_5 (mg/l)

Figure 90



SLUDGE GROWTH RATE vs. REMOVAL VELOCITY (BOD_5 BASIS)

Figure 9.1

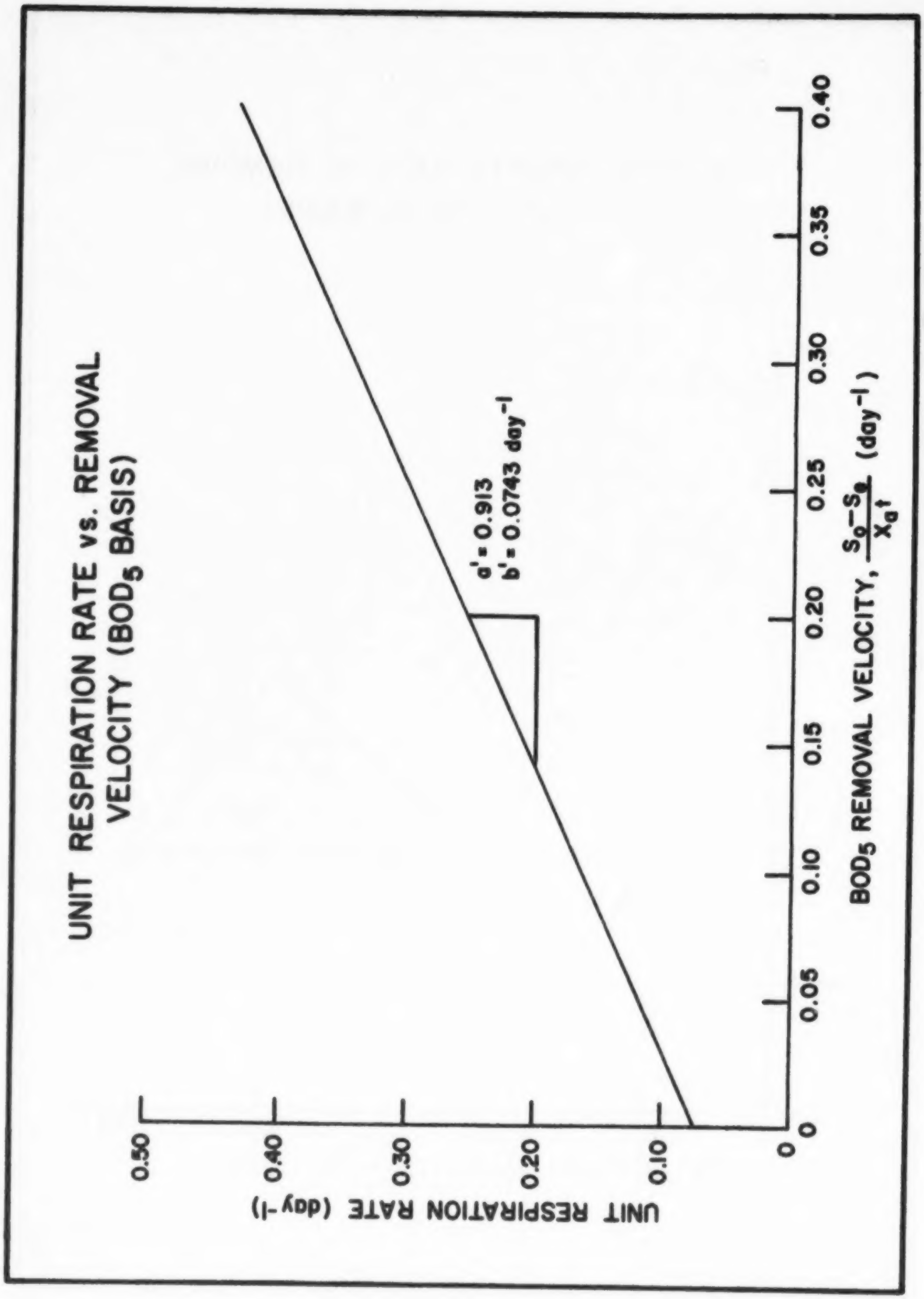

UNIT RESPIRATION RATE vs. REMOVAL VELOCITY (BOD_5 BASIS)
$a' = 0.913$
$b' = 0.0743\ day^{-1}$
UNIT RESPIRATION RATE (day^{-1})
0
0.10
0.20
0.30
0.40
0.50
BOD_5 REMOVAL VELOCITY, $\frac{S_o - S_e}{X_a t}$ (day^{-1})
0
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40

Figure 92

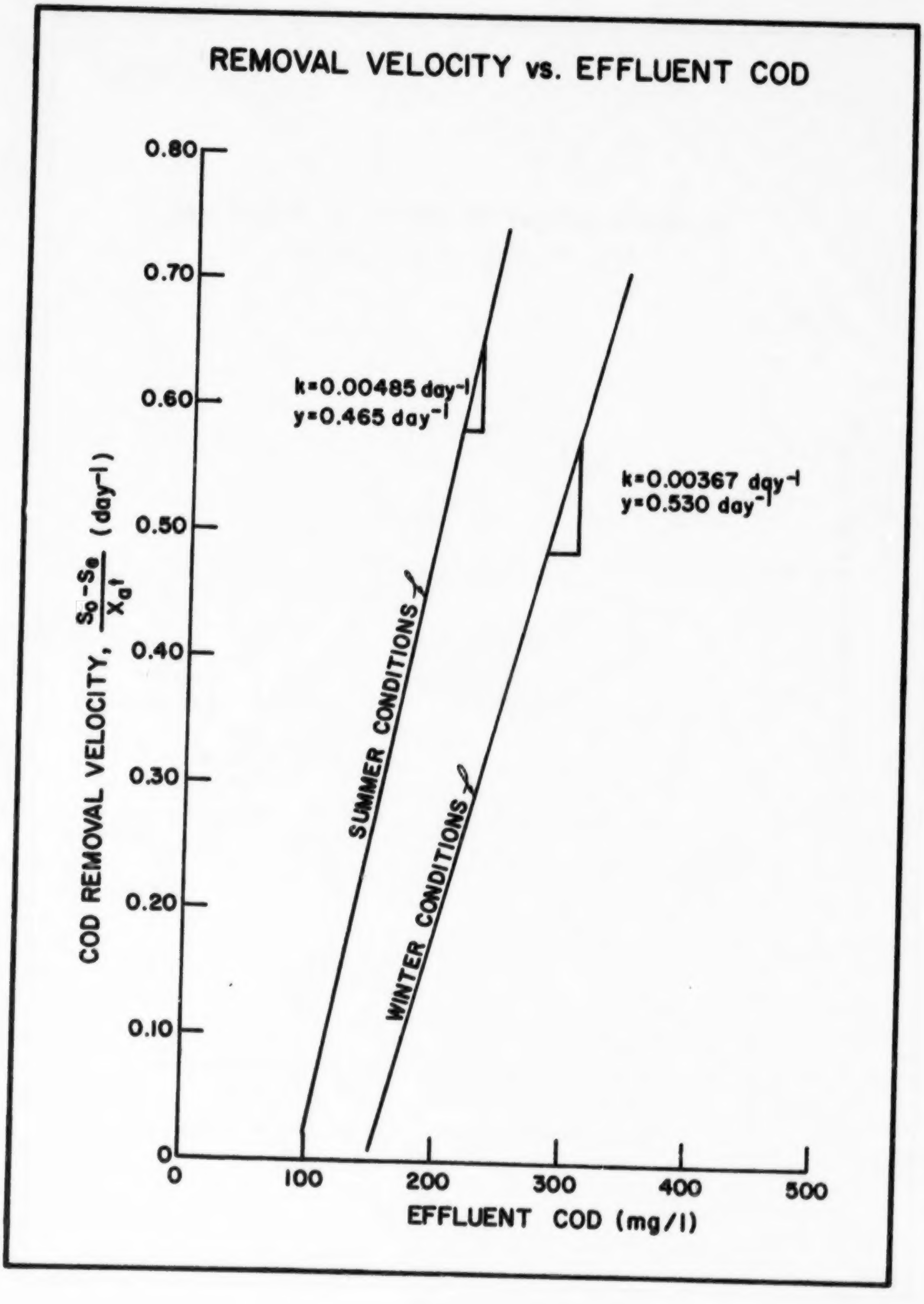
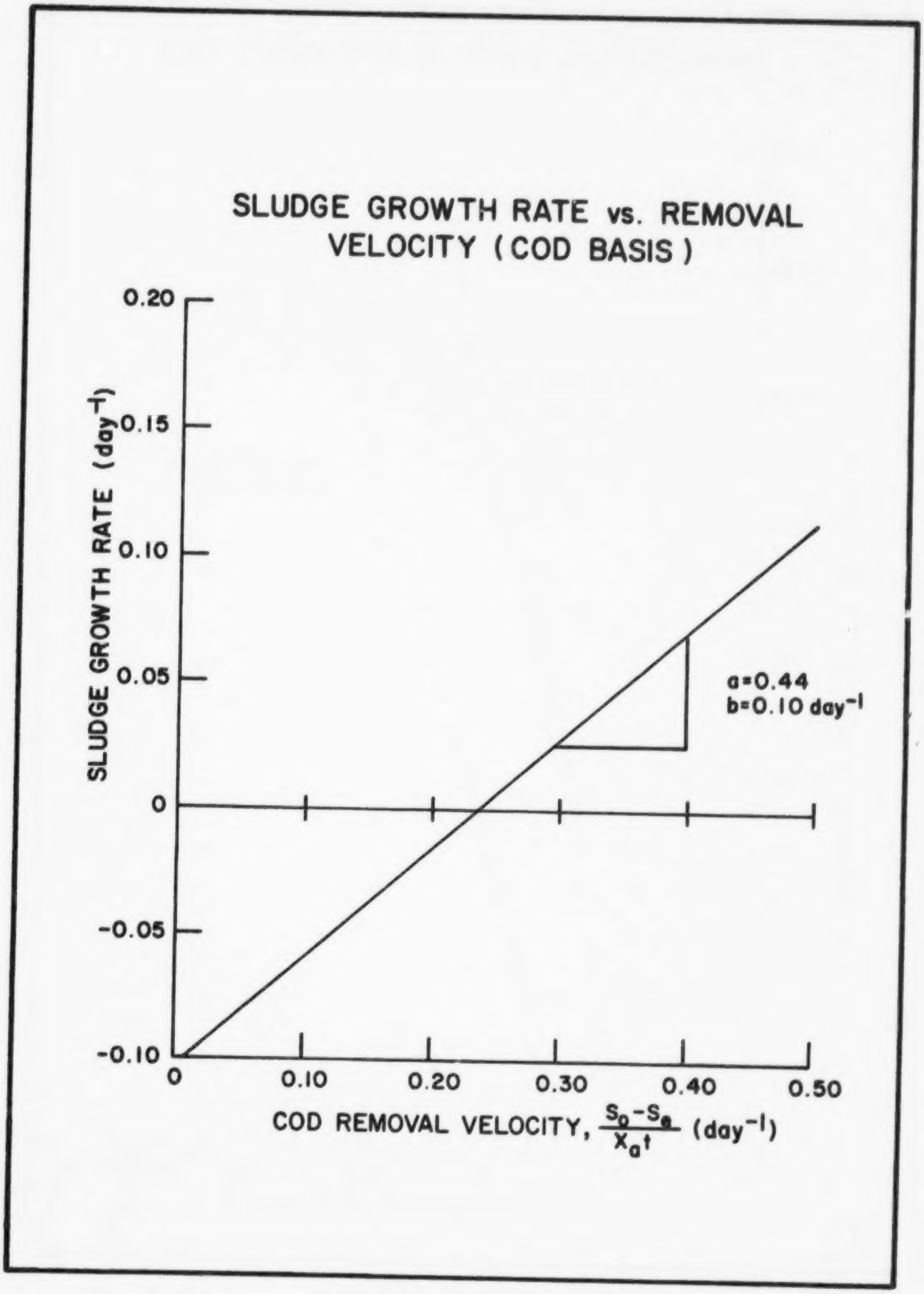
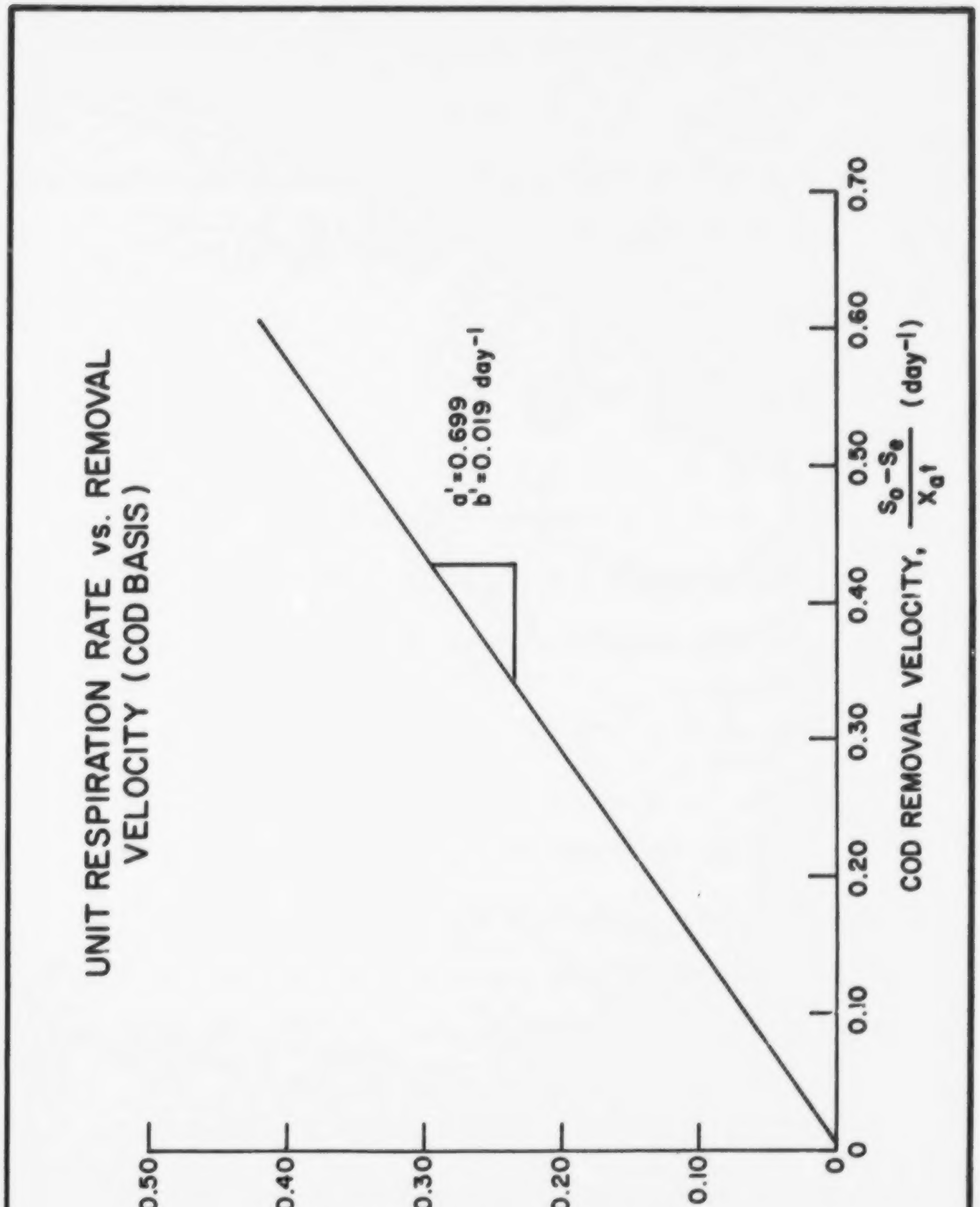

REMOVAL VELOCITY vs. EFFLUENT COD
COD REMOVAL VELOCITY, $\frac{S_o - S_e}{X_a t}$ (day^{-1})
EFFLUENT COD (mg/l)
0
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0
100
200
300
400
500
k=0.00485 day^{-1}
y=0.465 day^{-1}
k=0.00367 day^{-1}
y=0.530 day^{-1}
SUMMER CONDITIONS
WINTER CONDITIONS

Figure 93



SLUDGE GROWTH RATE vs. REMOVAL VELOCITY (COD BASIS)

Figure 94


UNIT RESPIRATION RATE vs. REMOVAL VELOCITY (COD BASIS)
a' = 0.699
b' = 0.019 day⁻¹
UNIT RESPIRATION RATE (day⁻¹)
0
0.10
0.20
0.30
0.40
0.50
COD REMOVAL VELOCITY, (So − Se)/Xa' (day⁻¹)
0
0.10
0.20
0.30
0.40
0.50
0.60
0.70

Therefore;

$$S_o = 230 - (0.2)\,(230) = 184 \text{ mg/l}$$

$$S_e = (230)\,(1 - 0.875) = 29 \text{ mg/l}$$

From the STATPK-developed relationship shown in Appendix A, Figure A-2;

$$t = \frac{(S_o - S_e)\,(24 \text{ hrs/day})}{X_a\,(K\,S_e - y)} = \frac{(184 - 29)\,(24 \text{ hrs/day})}{(1{,}500)\quad(0.0115)\,(29) - 0.125}$$

$$\underline{\underline{t = 12.0 \text{ hours}}}$$

(Winter Conditions - BOD basis)

Design Basis:

1. BOD of raw wastewater = 360 mg/l
2. BOD removal in primary clarifier = 10%
3. Total removal = 66% (observed efficiency during coldest month)
4. MLVSS = 2,000 mg/l

Therefore;

$$S_o = 360 - 0.1\,(360) = 324 \text{ mg/l}$$

$$S_e = 360\,(1 - 0.66) = 122 \text{ mg/l}$$

From the STATPK-developed relationship shown in Appendix A, Figure A-2;

$$t = \frac{(S_o - S_e)(24 \text{ hrs/day})}{X_a\,(K\,S_e - y)} = \frac{(324 - 122)\,(24 \text{ hrs/day})}{(2{,}000)\quad(0.00487)\,(122) - 0.125}$$

$$\underline{\underline{t = 5.2 \text{ hours}}}$$

Use a design detention time of 12 hours as summer conditions control.

Oxygen Requirements:

Calculated as lbs $0_2/10^6$ gal

Assume maximum condition - i.e., winter conditions

$S_o - S_e = S_r = 202$ mg/l $= 1,690$ lbs/10^6 gal

$X_a = 2,000$ mg/l assuming 12 hour aeration time
then $X_aV = 8,300$ lbs

Then:

$$R_rV \text{ lbs } 0_2/10^6 \text{ gal} = a'S_r + b'X_aV$$

$$= (0.913)(1,690) + (0.0743)(8,300)$$

$$= 1,540 + 624$$

$$= 2,164 \text{ lbs } 0_2/10^6 \text{ gal}$$

Sludge Production:

Calculated as lbs sludge/10^6 gal

Neglecting influent and effluent solids

$\Delta X = a\, S_rQ - b\, X_at$

Assume $S_rQ = (202 \text{ mg/l})(8.34)(1 \text{ MGD}) = 1,690$ lbs/10^6 gal

$X_at = (1,500 \text{ mg/l})(8.34)(0.5)(1 \text{ MGD}) = 6,250$ lbs

Then:

$$\Delta X = (0.445)(1,690) - (0.10)(6,250)$$

$$\Delta X = 753 - 625$$

$$\Delta X = 130 \text{ lbs sludge}/10^6 \text{ gal}$$

Use 500 lbs sludge/10^6 gal based on similar installations. Based on the above calculations, the following parameters would be applicable to the theoretical design of the regional plant using a flow of 72 MGD.

Required Detention Time = 12.0 hours

Total Oxygen Requirement: $\frac{2,164 \text{ lbs } O_2}{10^6 \text{ gal}}$ (72 MGD) = 155,000 lbs/day

Estimated Sludge Production (VSS, dry wt) = $\frac{500 \text{ lbs VSS}}{10^6 \text{ gal}}$ (72 MGD)

= 36,000 lbs/day

Summary (Biological Treatment)

The results of the pilot plant treatability studies have been presented herein. These results indicate that a biological system has the capacity to remove the organic constituents of the combined wastewater to a quality level acceptable in terms of BOD for discharge with the possible exception of cold weather operations. Additionally, the results indicate that equalization at the regional site is not required. The proposed biological system will include an aeration detention time of 12 hours and should be so designed to provide for a completely mixed flow regime. The observed effluent quality of the biological system is presented in the following Section and is compared to the effluent quality standards as set forth by the Delaware River Basin Commission.

PILOT PLANT PROCESS EVALUATION - SLUDGE HANDLING

Various methods of sludge dewatering were evaluated in the pilot plant treatability program. Included within these studies were dewatering methods such as centrifugation, filter pressing and vacuum filtration. Additionally, aerobic digestion of the biological solids was tested. The results of these evaluations are presented herein.

Aerobic Digestion

Stabilization of biological solids under aerobic conditions is often termed as aerobic digestion. The process is widely used to reduce the volatile fraction of waste solids from activated sludge systems and is most feasible when the volatile fraction of the suspended solids is greater than 60 percent. In cases where the volatile suspended solids is less than 50 percent, it is normally not practical to use this means of sludge treatment. During the process, oxygen is added under completely mixed conditions, and the biomass is reduced to carbon dioxide, water, and other end products with very little synthesis occurring. The process is often called "auto-oxidation" or "endogenous respiration." If primary sludge is introduced into the system, the synthesis and oxygen requirements must be increased to accommodate the additional load. After aerobic stabilization, the sludge may be concentrated and dewatered using sand drying beds, vacuum filters, filter presses, or centrifuges.

Procedure

Aerobic digestion was simulated on a pilot and bench scale level during the course of this work. In practice, the process is normally conducted on a fill and draw basis and thus the use of batch techniques is appropriate. The primary influent feed was shut off from one of the pilot plant aeration basins during the period of June through July, 1970, and the basin was operated as an aerobic digester. The bench scale studies consisted of setting up three 8 liter reactors which are shown in Section V, Figure 7. Each reactor was supplied with diffused air. Waste activated sludge was concentrated by gravity prior to being added to the reactors. During both the pilot and bench scale studies, the following analyses were made on the mixed liquor; total suspended solids, volatile suspended solids, oxygen uptake, and pH. Periodically, the BOD_5, COD, TOD, and pH, as well as phosphorus and nitrogen concentrations of the supernatant liquor were determined. Each of the bench scale reactors was operated for 20 days and during this period, no additional sludge was added.

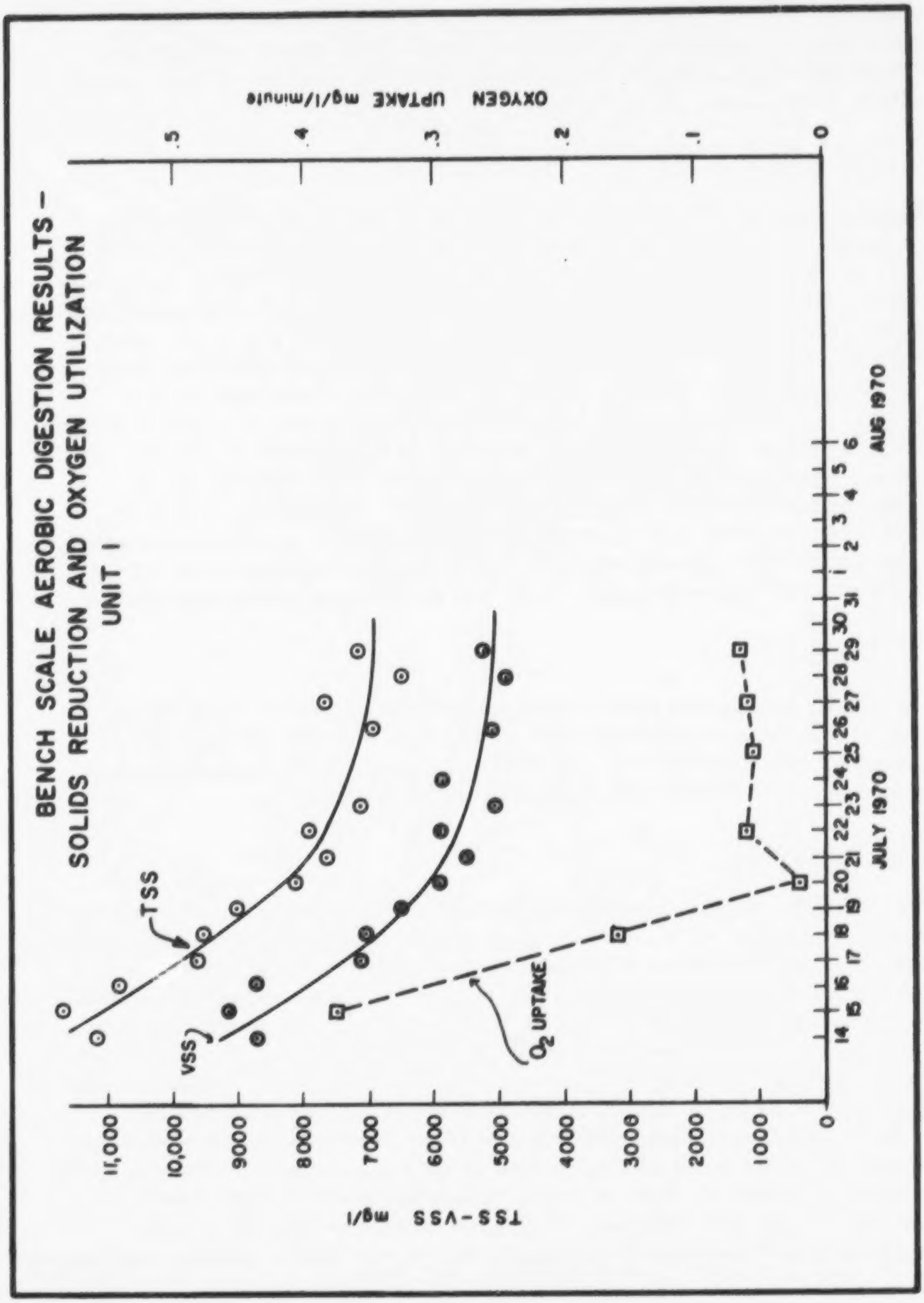
Results

The results, as measured by the suspended solids concentration of the reactor contents and oxygen uptake are presented in Figures 95, 96, 97, and 98. During the bench scale studies, three different initial solids concentrations were used. Stabilization efficiencies are shown in Figure 99.

Summary

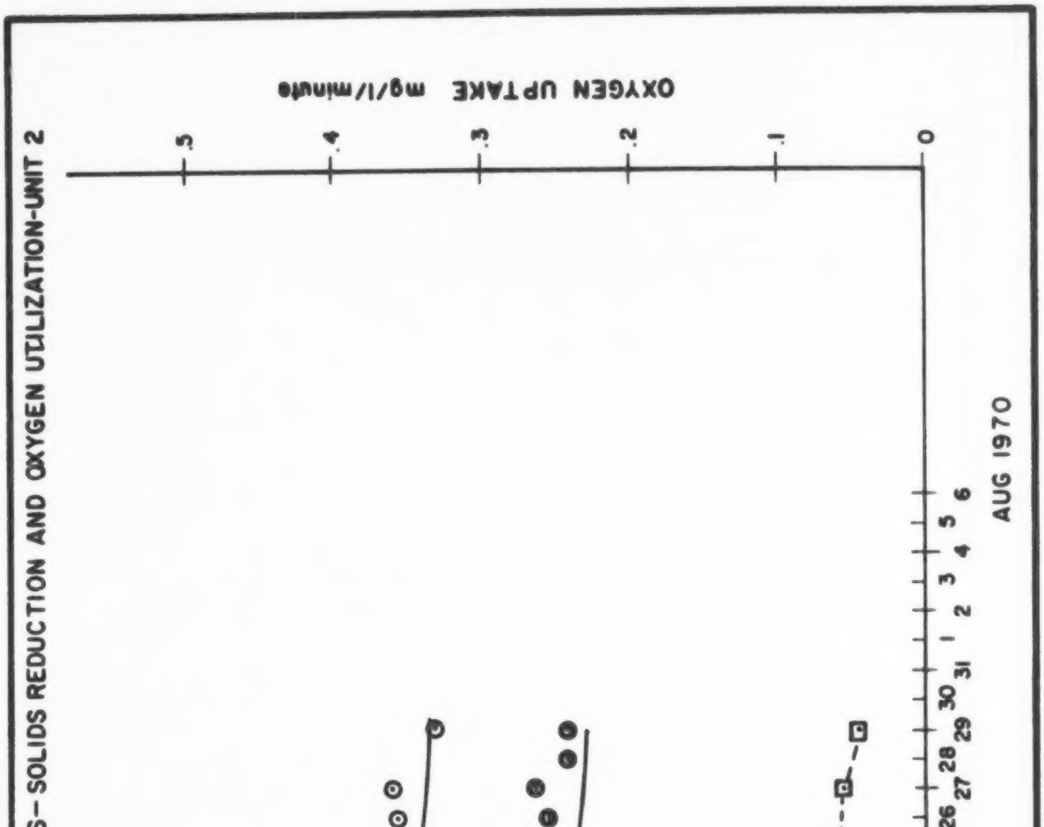
The data indicate that a maximum of 50 percent VSS reduction could be achieved in 20 days and that up to a concentration of one percent solids, the solids loading does not affect the rate of stabilization. Approximately 50 percent of the volatile solids are not removable during any realistic aeration period as reflected by the data during the last 10 days of aeration. The low oxygen utilization also indicates a low rate of cellular destruction through oxidation. This underscores the importance of thickening either in the digester or prior to digestion in order to achieve economy in design. A detention time of seven days should be sufficient to achieve 75 percent reduction of the digestible solids provided the reactor has facilities for continuous supernating and subsequent thickening of the contents. The aerobic digestion could be accomplished in earthen basins provided with surface aeration or in concrete basins provided either with mechanical or diffused aeration systems. Mixing will control aeration requirements

Figure 95



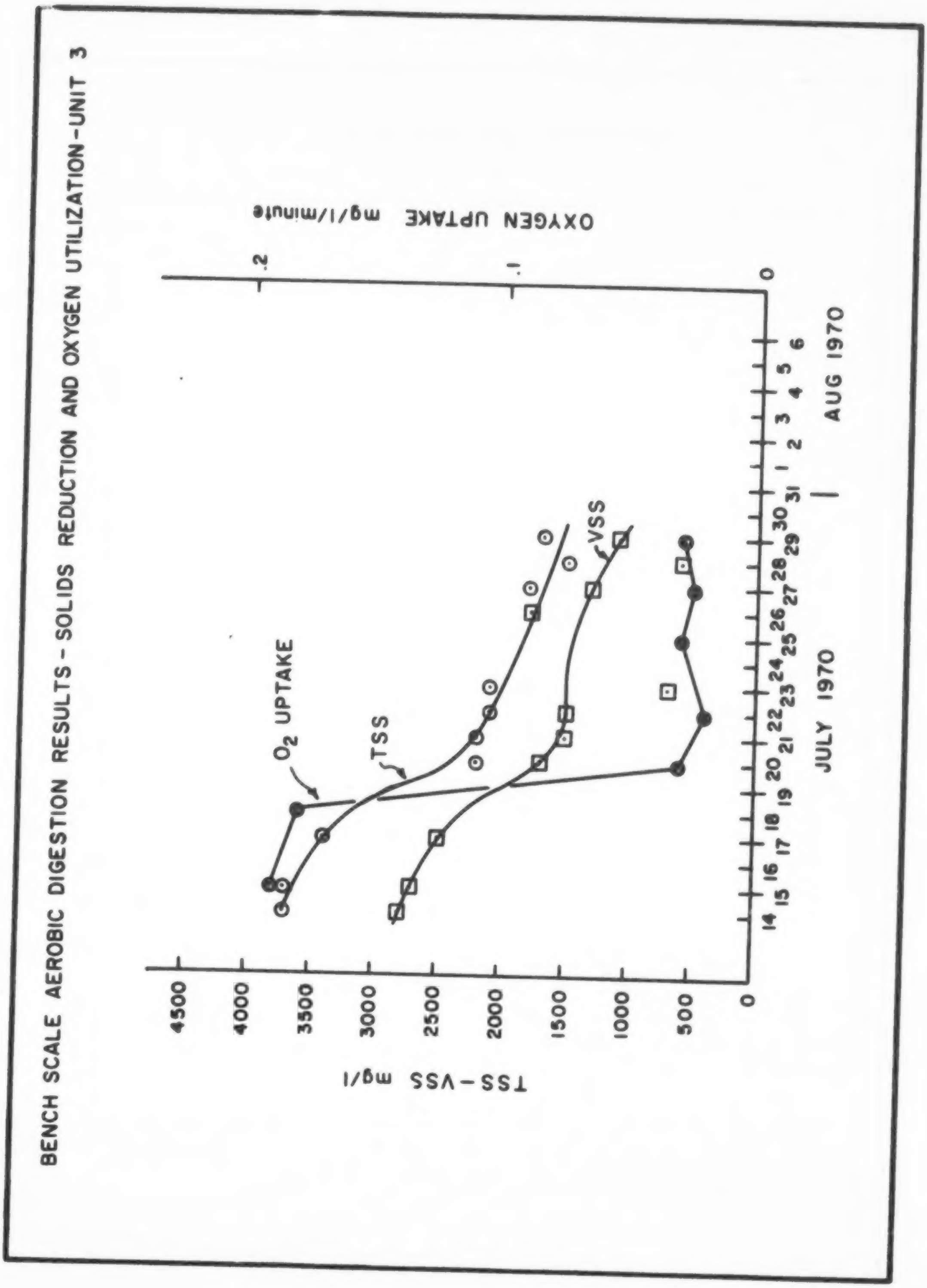
BENCH SCALE AEROBIC DIGESTION RESULTS — SOLIDS REDUCTION AND OXYGEN UTILIZATION
UNIT I

Figure 96



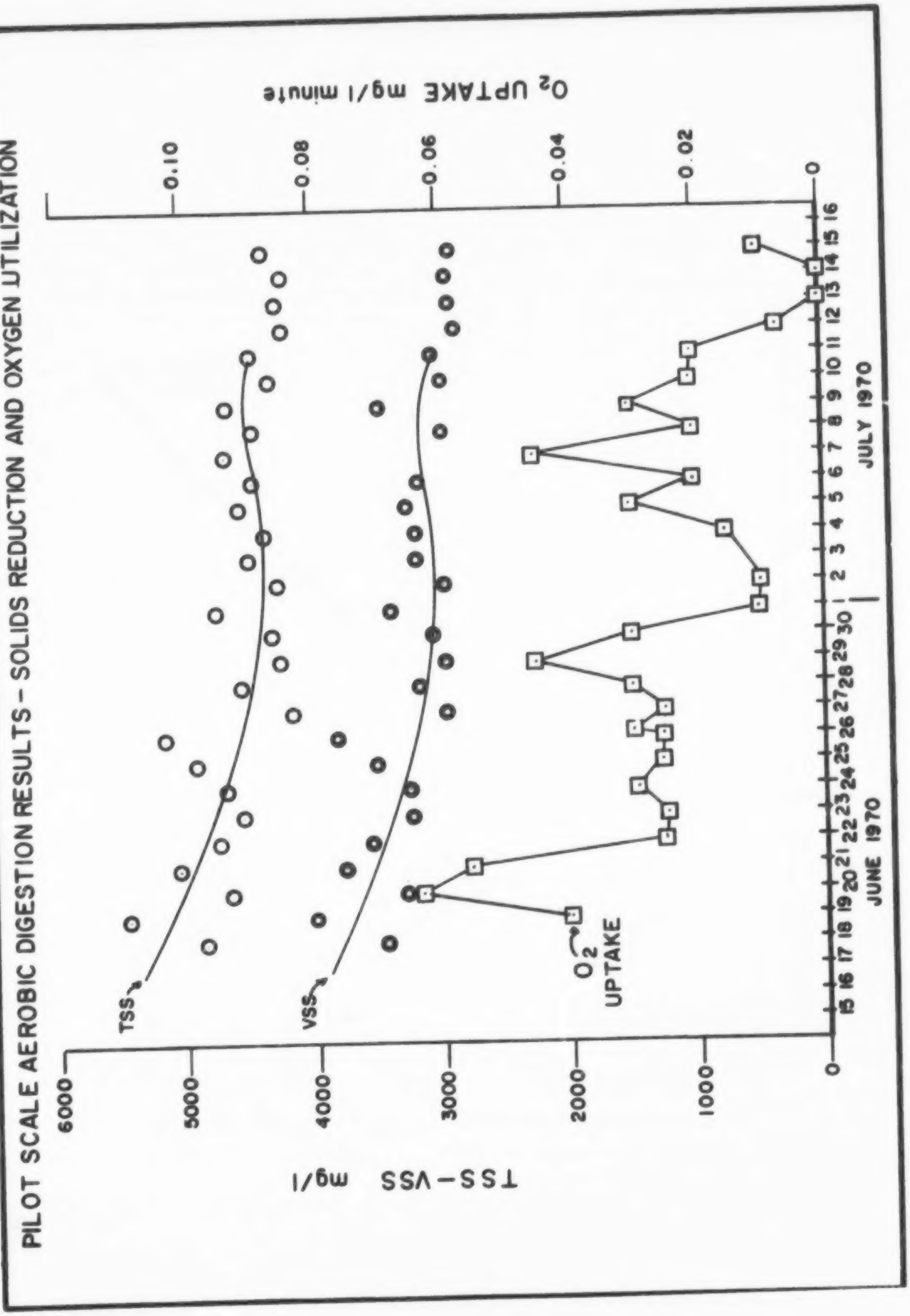
BENCH SCALE AEROBIC DIGESTION RESULTS—SOLIDS REDUCTION AND OXYGEN UTILIZATION-UNIT 2

Figure 97



BENCH SCALE AEROBIC DIGESTION RESULTS - SOLIDS REDUCTION AND OXYGEN UTILIZATION-UNIT 3

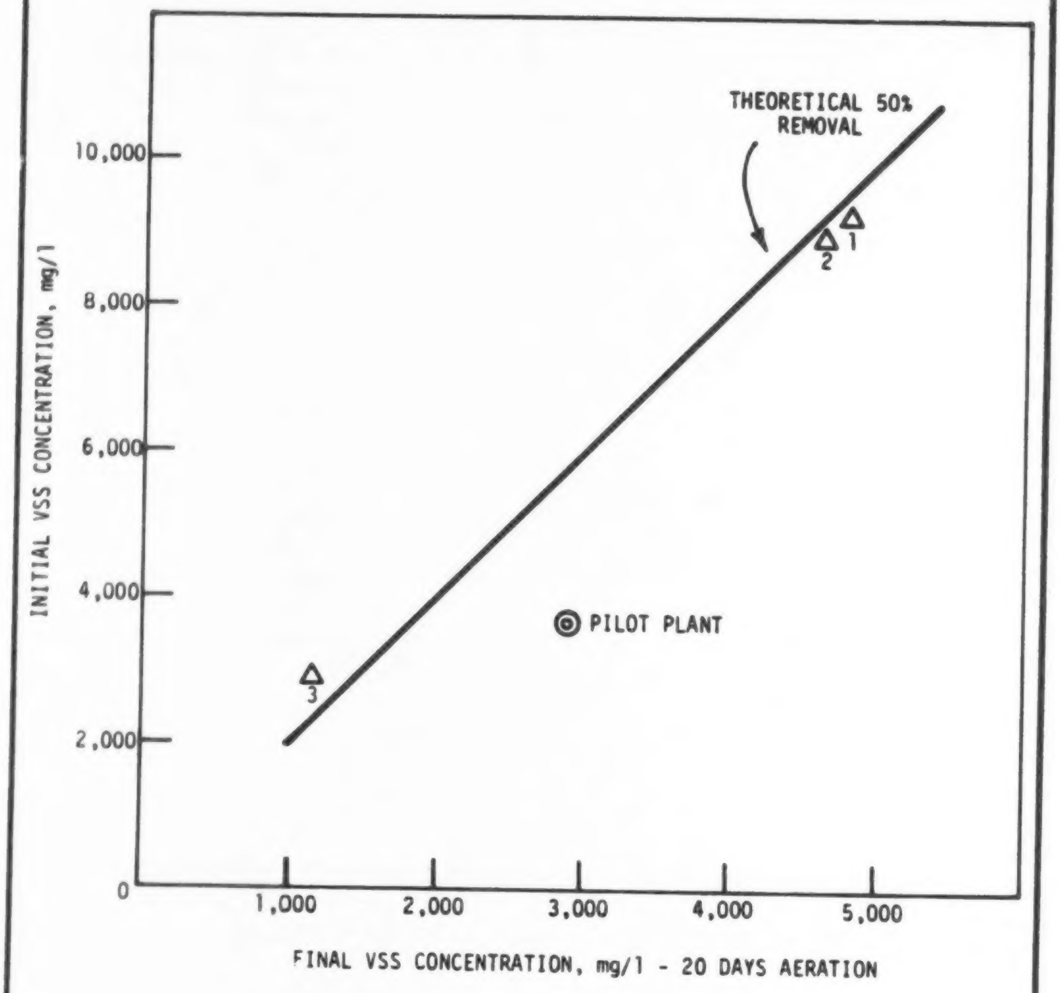
Figure 98



PILOT SCALE AEROBIC DIGESTION RESULTS – SOLIDS REDUCTION AND OXYGEN UTILIZATION

Figure 99

AEROBIC STABILIZATION OF VOLATILE SOLIDS



and therefore the aeration system should be designed on a basis of approximately 0.15 HP per thousand gallons of aeration volume in the case of mechanical aerators.

Filter Press

The fixed plate high pressure filter press may be used to dewater waste sludges produced by municipal water and wastewater treatment facilities as well as industrial sludges. The process produces filter cakes containing up to 55 percent solids which are suitable for land disposal or incineration. The economics of the process are enhanced by thickening prior to pressing and by utilizing incinerator ash as a conditioner. The press does not dewater solids by squeezing, but operates similar to a rotary vacuum filter, except higher pressures are used.

Procedure

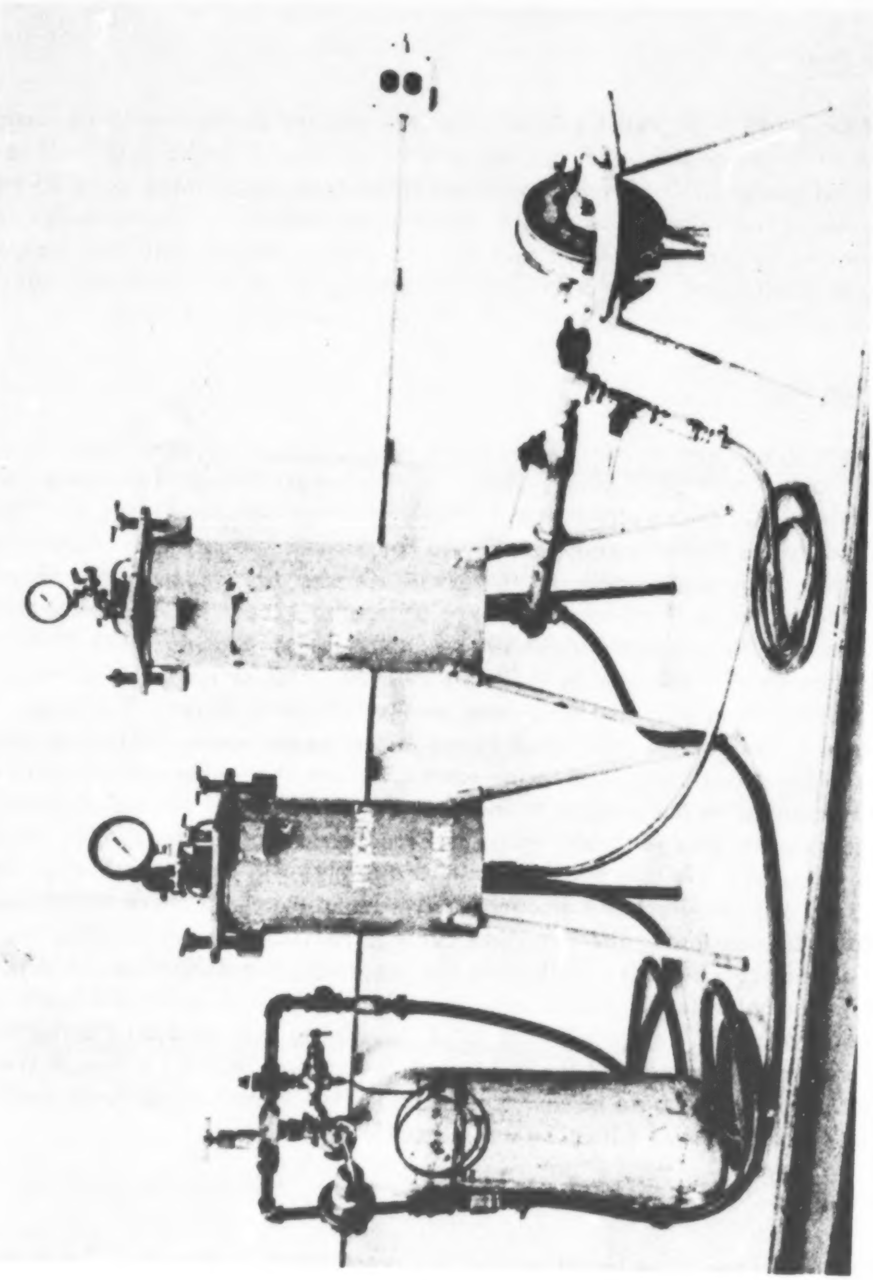
The filter press process may be simulated by the use of the filter press "bomb" or a larger pilot plant. However, the larger facility requires a considerable quantity of waste sludge. During these studies, a filter press "bomb" supplied by Beloit-Passavant was used to investigate the process. Illustrations of the pilot apparatus are shown in Figures 100 and 101. Three types of waste solids generated at the pilot plant were utilized for these investigations and in all cases the sludges were thickened by gravity before testing. The filter press bomb consists of a nitrogen or CO_2 gas cylinder pressure source, a pre-coat tank, filter feed tank, and a six inch nominal diameter filter. The press produces a cake about 3/4" thick in the center which tapers off toward the outer edges. The process has been found more efficient at an elevated pH and therefore lime was added to the sludges to increase the pH to above 10. In addition, the process requires sludge conditioning and for these investigations, diatomaceous earth was used. No attempt was made to optimize the quantity of body feed required, and therefore the amounts used were in excess of those which would normally be required. The filtration cycle is preceded by pre-coating the press with diatomaceous earth. Following the pre-coat, the sludge and conditioning material combination is pressed onto the filtering medium and the filtrate is forced through the center with the solids remaining in a cake on the filter. The maximum pressure used for this investigation was 340 psi, although the normal operating pressure would be around 230 psi. The operation was continued for 30 minutes at which time filtration was virtually at a standstill.

Results

The results of the pilot investigations are presented in Table 39. The data indicates moisture contents in excess of 60 percent resulting in a total solids concentration of about 38 percent. However, approximately half of the solids content of the cases

Figure 100

PILOT FILTER PRESS ASSEMBLY



FILTER PRESS ASSEMBLY

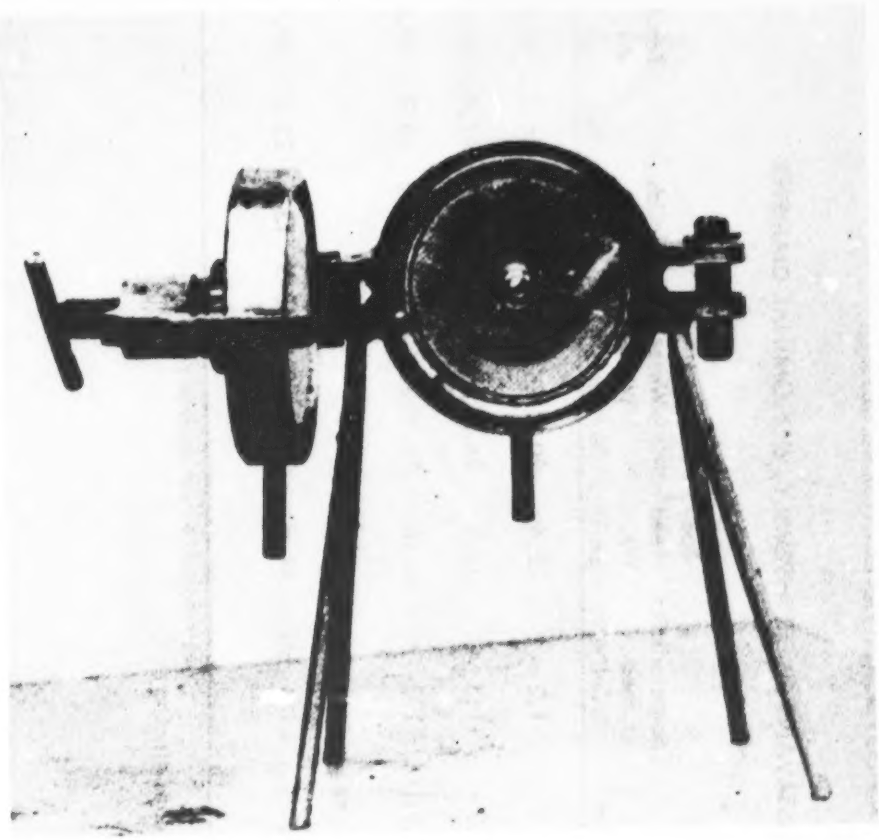


TABLE 39

PILOT SCALE FILTER PRESS RESULTS

BELOIT-PASSAVANT FILTER ASSEMBLY, 6" NOMINAL DIAMETER

WASTE SOLIDS	pH[2]	Characteristics of Feed % Solids	Body Feed[1] Wt. gm	%	Water Wt. gm	%	Solids Wt. gm	%	Total Wt. gm	Filtrate Volume ml
Activated	9.5	1.2	12.6	20.7	38.0	62.5	10.3	16.7	60.9	7000
Digested Activated	11.3	1.5	14.9	24.7	36.8	60.5	10.7	17.6	62.4	7800
Primary	10.9	1.3	11.6	19.1	38.9	64.0	9.3	15.3	59.8	7200
Primary and Activated	12.1	1.4	14.4	23.7	39.9	65.5	15.6	25.7	69.9	7100

---

[1] Diatomaceous earth added for conditioning

[2] Lime added for pH adjustment

was diatomaceous earth. The best results in terms of the filtrate volume were obtained using the digested waste activated sludge. However, the filter cake produced from primary and activated sludge contained the greatest percentage of sludge solids. It is noteworthy that the pH of this sludge combination was the highest at 12.1. The highest concentration of feed solids before the addition of diatomaceous earth was 1.5 percent; however, in practice, these concentrations might be increased to two to three percent, thereby enhancing the process.

Summary

In summary, the filter press results reflected the highest solids concentrations obtainable when compared to other dewatering processes which were simulated. However, it is important to recognize that a great portion of the solids concentration consisted of conditioning chemicals. If incineration is not included in the sludge disposal system, ash or other conditioning chemicals must be provided. For the most part, the dewatered sludge volumes obtained in the filter press would be less than those obtained by using other means; however, the weight in most cases would be greater.

Filter Leaf

Sludge filtration studies using a filter leaf apparatus were conducted in order to predict sludge yield values for specified operating conditions. Primary sludge, excess activated sludge, and a combination of the two were used. Although other sludge dewatering modes were tested more extensively on a pilot scale, the filter leaf sludge filtration approach provides useful information with respect to the effect of operating variables on dewaterability. Moreover, the practicality of using vacuum filtration methods for dewatering the sludges in question can be assessed.

Samples of primary and excess activated sludge accumulated in the normal operation of the pilot plant were thickened and taken into the laboratory for filter leaf testing. The filter leaf apparatus used in this experiment is shown in Figure 102. The predictive equation for filter performance is:

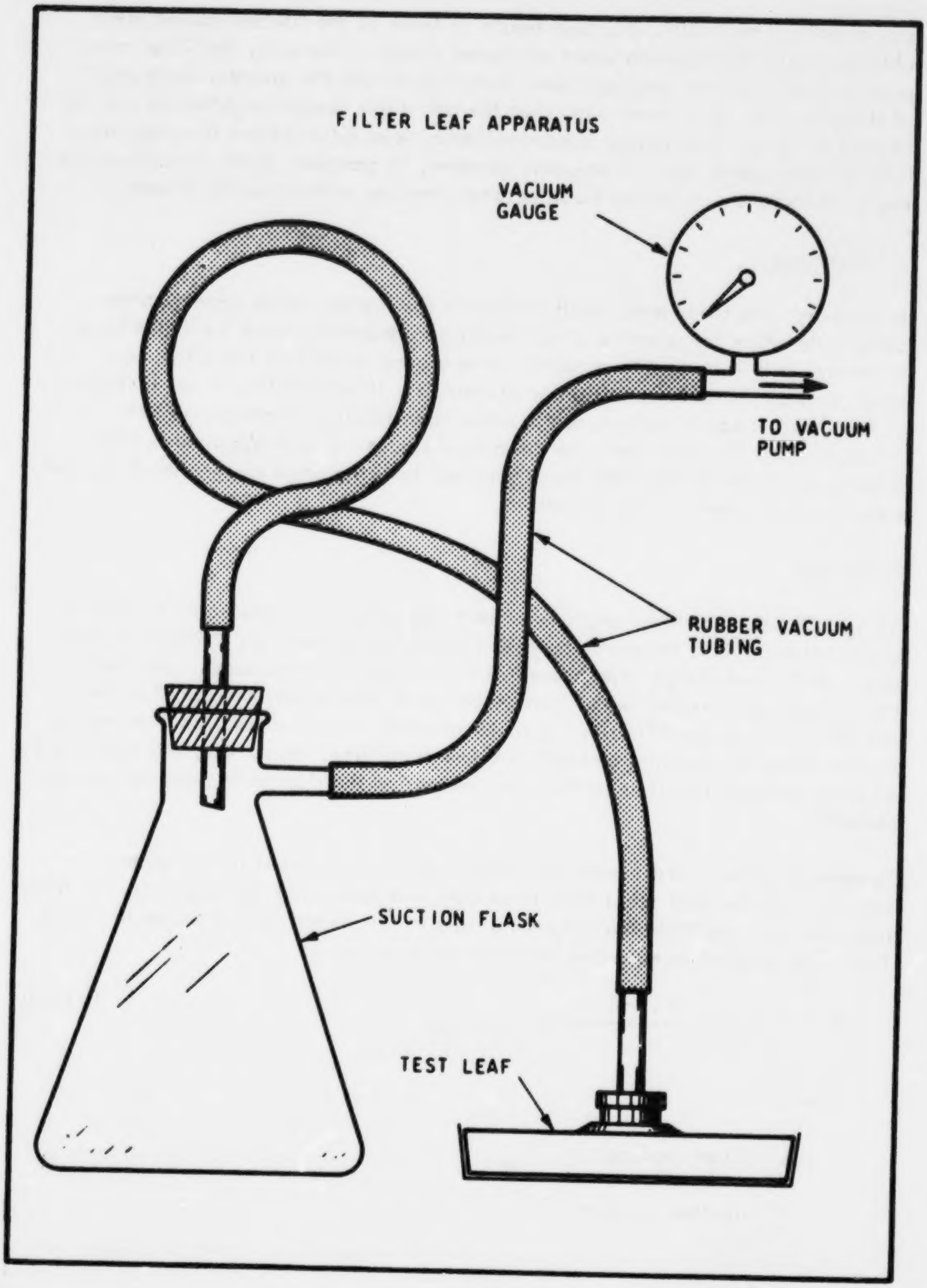
$$L = 35.7 \left[\frac{P(1-s)}{\mu R_o} \right]^{1/2} \frac{C^m}{t_f^n} \qquad \text{(VI-13)}$$

where:

L = filter loading

P = applied vacuum

Figure 102


FILTER LEAF APPARATUS
VACUUM GAUGE
TO VACUUM PUMP
RUBBER VACUUM TUBING
SUCTION FLASK
TEST LEAF

μ = filtrate viscosity

R_o = filter resistance

C = solids deposited per unit volume of filtrate

t_f = form time

The leaf test studies were directed toward the determination of the empirical constants (1-s), m, and n, as these exponents vary according to the nature of the sludge. These constants were evaluated by measuring the sludge yield as a function of operating vacuum, form time, and initial solids concentration. A bleed valve on the vacuum pump enabled vacuum control. Form time was obtained by submerging the leaf apparatus in the test sludge beaker for prescribed periods of time. The initial solids concentration was varied by diluting with sludge filtrate. The procedure used in the performance of this task is outlined elsewhere (Reference 11).

Data Analysis

The test results for each sludge run are tabulated in Table 40. These data in turn are plotted in Figures 103, 104, and 105 with the value of the constants for each sludge noted on the plot. The filter loading values are calculated on the basis of form time.

Summary

As indicated by the data, the unconditioned primary sludge, either alone or combined with excess activated sludge, was not amenable to rapid or effective dewatering based on the filter leaf test results. It is recognized that the yield could be enhanced to some extent by the addition of coagulant aid. The excess activated sludge, however, exhibited higher sludge yields and appears to be more amenable to vacuum filtration.

The data as presented herein can be used in sizing vacuum filtration units for the prototype treatment system. Based on the resolution of observed data, Equation VI-13 can be used for the general sizing of units, applying the following exponents as shown in Table 41:

Figure 103

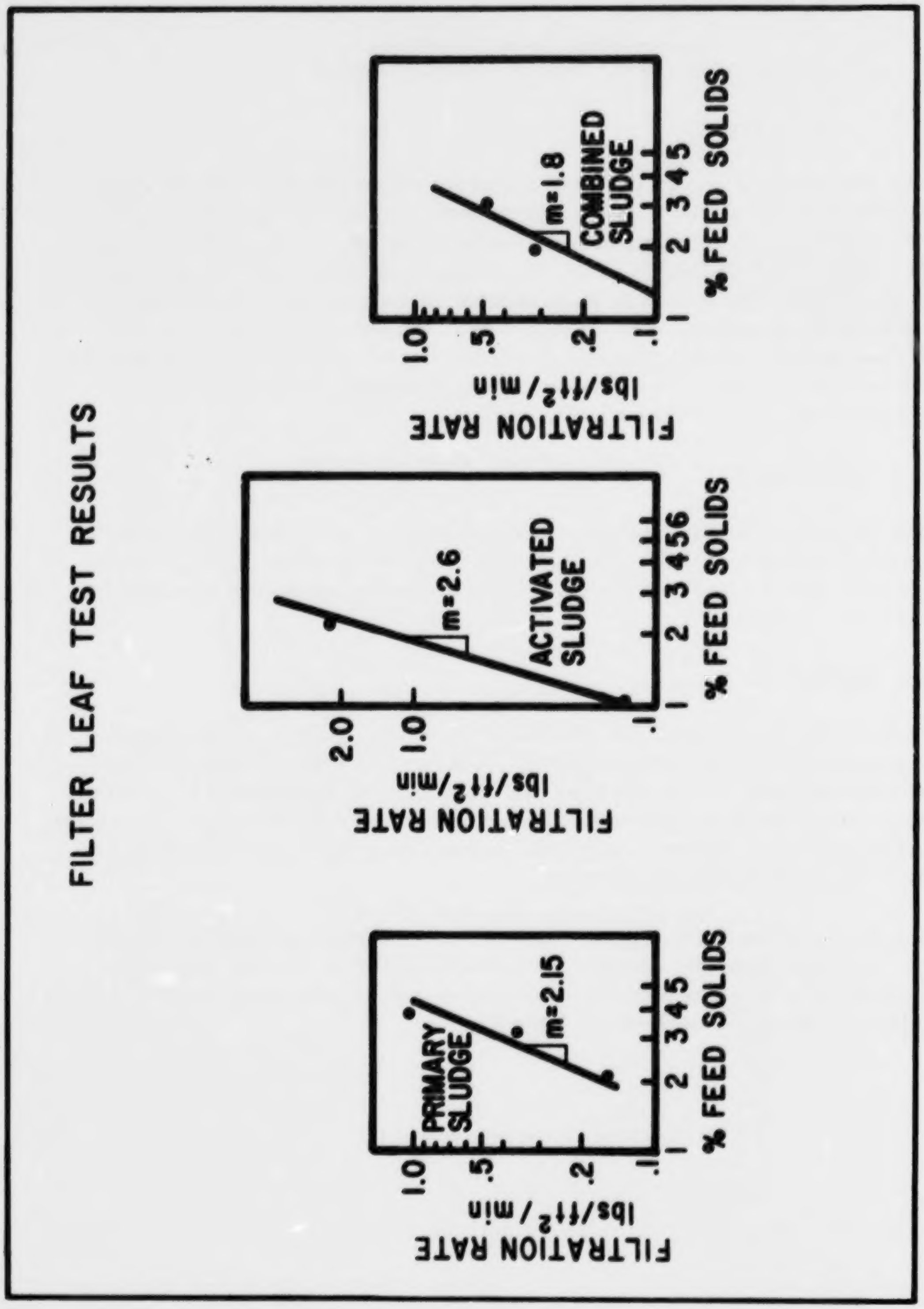

FILTER LEAF TEST RESULTS

TABLE 40

VACUUM FILTRATION STUDIES

Sample No.	Initial Solids (mg/l)	Vacuum (in. Hg.)	Form Time (min.)	Dry Time (min.)	Yield (gms., dry wt.)	Moisture (% solids)
Run No. 1 - Primary Sludge						
1	36,240	20	2	1.5	1.659	30.0
2	36,240	20	5	1.5	1.790	27.4
3	36,240	20	10	1.5	3.330	23.8
4	36,240	12	2	1.5	1.312	26.0
5	36,240	6	2	1.5	1.080	25.0
6	22,000	20	2	1.5	0.565	26.4
7	15,520	20	2	1.5	0.321	20.0
Run No. 2 - Activated Sludge						
1	21,280	20	2	1.5	3.314	9.9
2	21,280	20	5	1.5	3.408	9.9
3	21,280	20	10	1.5	3.911	12.5
4	21,280	12	2	1.5	2.748	10.5
5	21,280	6	2	1.5	2.938	10.7
6	12,160	20	2	1.5	0.232	12.6
7	13,600	20	2	1.5	0.064	13.9
Run No. 3 - Primary-Activated Sludge						
1	26,120	20	2	1.5	0.758	23.2
2	26,120	20	5	1.5	0.827	24.0
3	26,120	20	10	1.5	2.806	16.7
4	26,120	12	2	1.5	0.126	4.5
5	26,120	6	2	1.5	0.090	24.5
6	20,800	20	2	1.5	0.636	25.6
7	14,960	20	2	1.5	0.169	18.6

Figure 104

FILTER LEAF TEST RESULTS

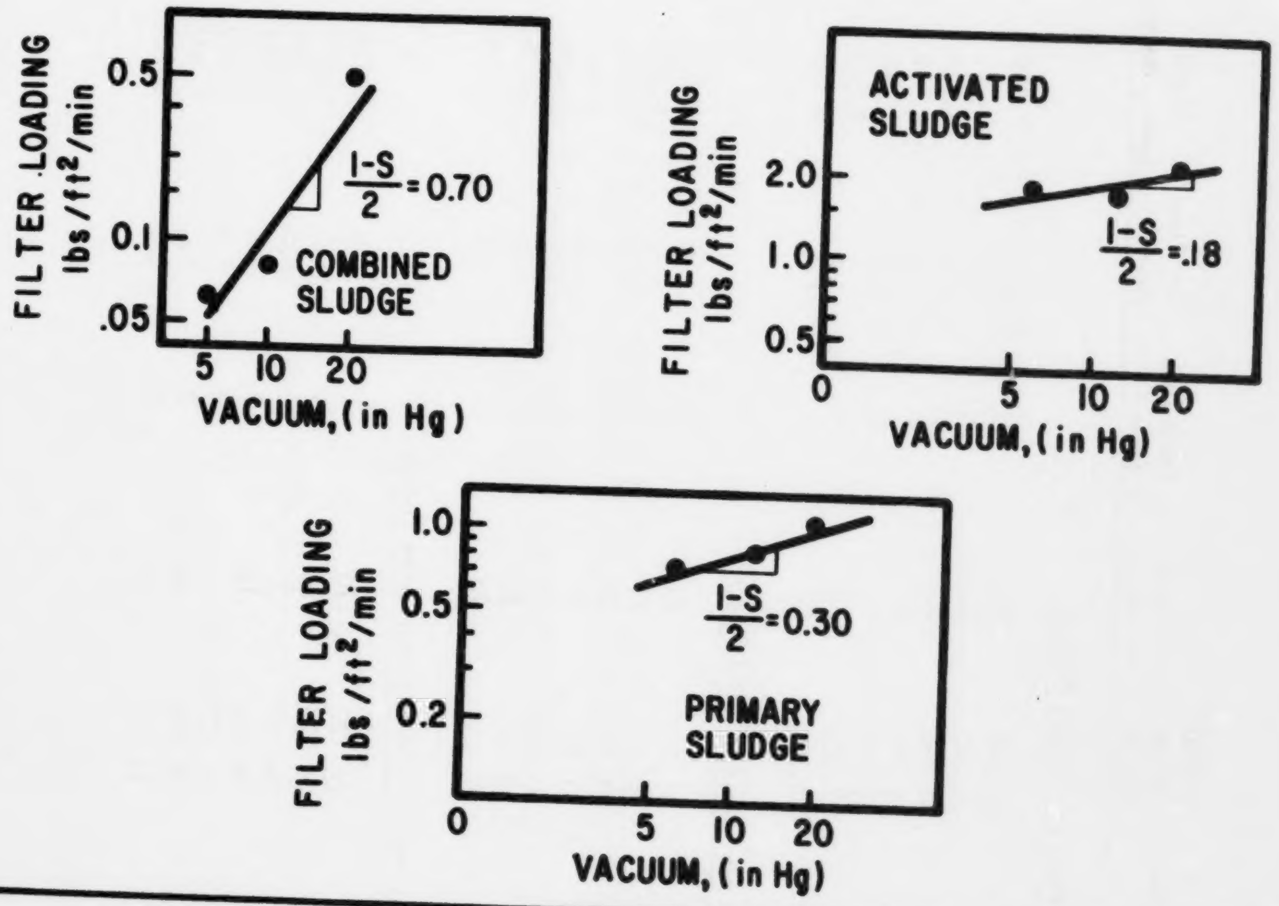


Figure 105

FILTER LEAF TEST RESULTS

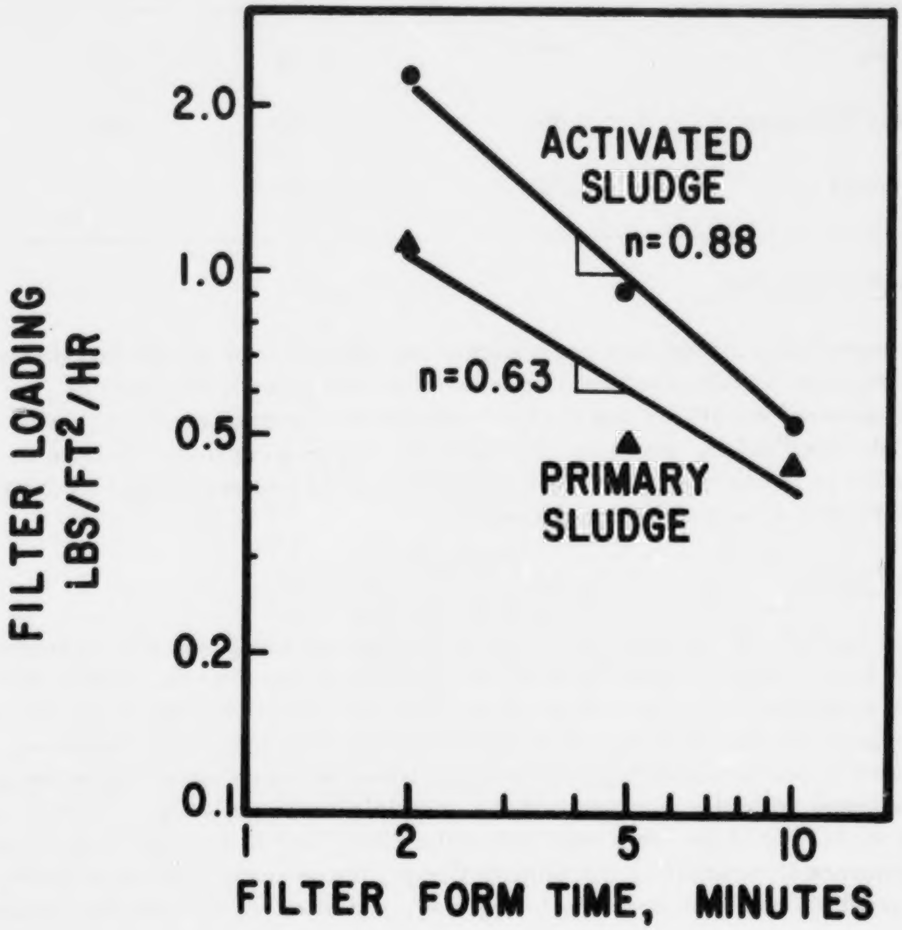


TABLE 41

VACUUM FILTRATION CONSTANTS

Sludge	$(\frac{1-s}{2})$	(m)	(n)
Primary	.30	2.15	.63
Excess Activated	.18	2.60	.88
Combined	.70	1.80	--

Sludge Drying Beds

Although sludge drying beds are a widely applied means of dewatering sludges, it is believed at this time that area constraints and other factors such as environmental conditions and sludge characteristics preclude serious consideration of their installation. However, if subsequent engineering studies dictate their inclusion in the system, pilot scale evaluation is not deemed mandatory for the development of process design information.

Centrifugation

Centrifugation, in general, is the use of mechanical equipment that separates solids from a liquid by sedimentation utilizing centrifugal force. Within the waste treatment field, centrifuges have been used effectively for sludge thickening and sludge dewatering with and without chemical addition. It is, however, difficult to predict centrifuge performance based on bench scale studies because of the many variables involved and the uncertainties in scale-up. For this reason, pilot scale centrifuges were installed at the Pilot Plant to evaluate centrifugal performance in dewatering the primary sludge, the secondary waste activated sludge which had been aerobically digested, and mixtures of these two sludges.

Three types of centrifuges were rented from the Sharples Division of the Pennwalt Chemical Corporation and included a Sharples P-600 Super-D-Canter solid bowl type centrifuge, a Sharples DHL Nozljector disc type centrifuge and a 14" Fletcher solid bowl basket type centrifuge as indicated in Table 42 and Figure 106. Each of these units was skid-mounted and equipped with the necessary electrical gear for operation. In addition, equipment on hand at the Pilot Plant was used as necessary for feed systems, sludge storage and chemical addition.

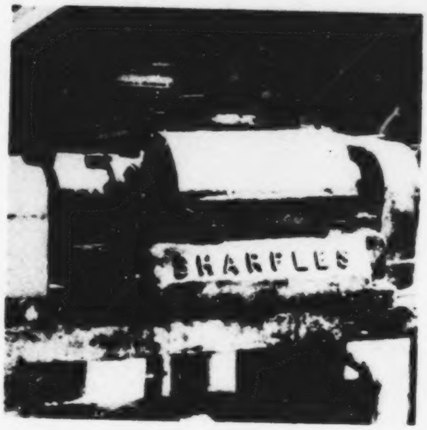
TABLE 42

CENTRIFUGES TESTED AT THE PILOT PLANT

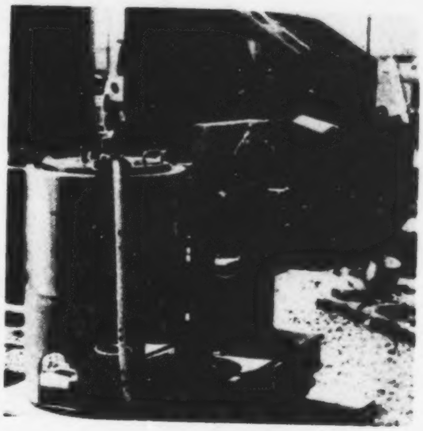
Model	Type	Sludges Tested
1. Sharples P-600 Super-D-Canter	Solid Bowl, Scroll	A) Primary B) 50/50 combination of primary and secondary aerobic digested. C) 75/25 combination of primary and secondary aerobic digested D) Secondary aerobic digested
2. Sharples DHL Nozljector	Disc type	A) Secondary digested B) 75/25 combination of primary and secondary aerobic digested
3. 14" Fletcher	Solid bowl with skimmer	A) Primary B) Effluent from P-600 on 75/25 combination of sludges

Figure 106

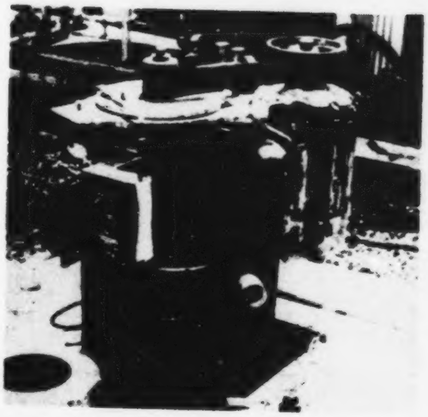
PILOT SCALE CENTRIFUGES



P-600



14" FLETCHER



DHL NOZLJECTOR

CHEMICAL FEED SYSTEM

Sharples P-600 Super-D-Canter Centrifuge

The Sharples P-600 centrifuge is a conventional type horizontal, cylindrical-conical, solid bowl machine in which the sludge is fed through a stationary feed tube along the center of the bowl to the hub of the screw conveyor. The screw conveyor is mounted inside the rotating bowl and rotates at a slightly lower speed than the bowl with the use of a planetary gear arrangement. Sludge leaves the end of the feed tube, is accelerated, passed through the ports of the conveyor shaft, and distributed to the periphery of the bowl. The solids are settled through the liquid and are moved along the bowl wall by the blades of the screw conveyor. The solids move out of the liquid bowl and onto a conical drainage deck and then are continuously conveyed by the screw to the end of the machine and discharged. The liquid effluent is discharged through effluent ports after traveling the length of the pool under centrifugal force. The depth of the liquid or pool volume can be varied by adjustment of weir plates located at the opposite end of the bowl. In addition, the P-600 centrifuge has a conveyor designed to add flocculent internally to the bowl so that the effects of these chemicals can be maximized.

In testing the solid bowl type centrifuge, several independent and dependent variables must be evaluated including the speed of rotation of the bowl, the speed of the conveyor with respect to the bowl, the liquid throughput, the solids throughput, the pool depth, the conveyor pitch, and the amount of flocculent added. The P-600 centrifuge was designed such that all of these variables could be evaluated on a pilot scale.

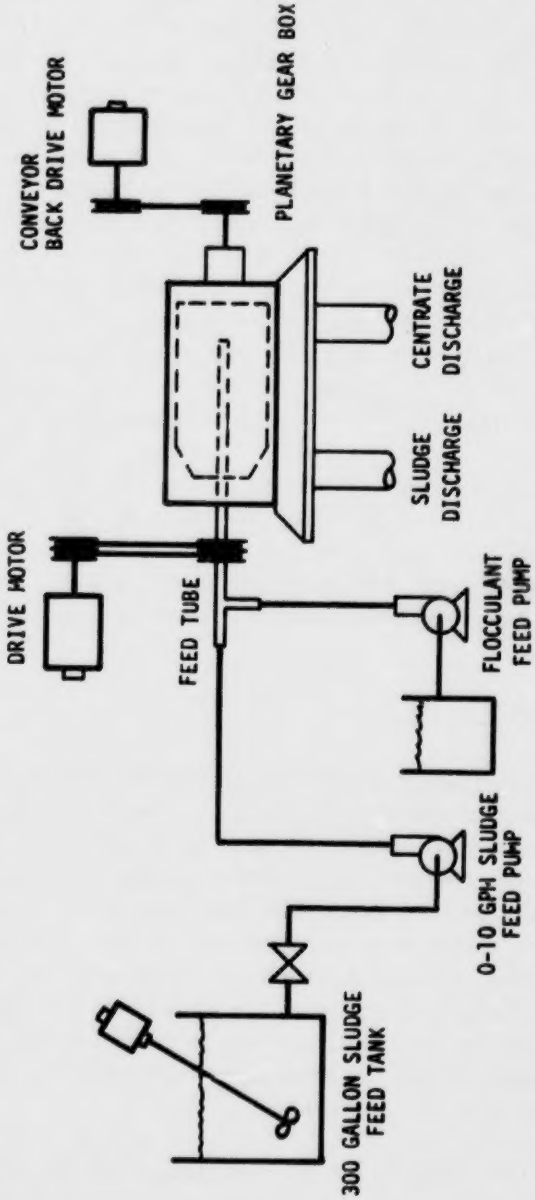
Procedure:

The P-600 centrifuge was installed to provide maximum flexibility in the testing program as shown in Figure 107. Prior to each test run, the rotation speed, the backdrive speed, the pool level and the conveyor pitch of the centrifuge were pre-set. A composite sample of sludge was pumped to the 300 gallon sludge feed tank. The centrifuge was brought up to operational speed and the sludge feed pump started. A minimum equilibrium time of ten (10) minutes was allowed for each run before samples were taken of the centrifuged sludge and centrate. In some cases the flow and flocculent feed were varied while the centrifuge was in operation thus allowing several tests to be completed during the same centrifuge run. The samples were analyzed for solids and moisture content.

Each of the two sludges, the primary sludge and the aerobically digested sludge, were tested individually with and without flocculent aids. Additionally, 50/50 and 75/25 percentage combinations of the primary and secondary sludge were evaluated. These various combinations of sludge were tested to provide additional design information for several alternate ultimate sludge disposal systems.

Figure 107

FLOW DIAGRAM FOR PILOT SCALE P-600 CENTRIFUGE


CONVEYOR BACK DRIVE MOTOR
PLANETARY GEAR BOX
CENTRATE DISCHARGE
SLUDGE DISCHARGE
DRIVE MOTOR
FEED TUBE
FLOCCULANT FEED PUMP
0-10 GPH SLUDGE FEED PUMP
300 GALLON SLUDGE FEED TANK

Results:

The results of the P-600 Centrifuge tests are shown in Figures 108 through 112. Figure 108 presents the results of the primary sludge with and without the addition of a flocculent. If the flocculent dosage is increased, the percent recovery increased accordingly. Figure 109 presents the same data on the aerobically digested sludge. Again, the percent recovery increases with flocculent dosage. Figure 110 presents the results of the combination of the two sludges indicating that the digested sludge has a higher recovery than does the primary sludge. Figure 111 and 112 present the 75/25 and 50/50 combination of the two sludges with and without flocculent addition at varying pool levels.

Fletcher Centrifuge:

The 14" Fletcher solid bowl basket type centrifuge consists of a vertical cylinder with a sludge storage capacity proportional to the height of the lip ring of the basket. The sludge is fed into the center of the bowl and is retained in the outer periphery with the centrate passing over the lip plate. The operation is batch type in that when the basket is full of centrifuged sludge, the feed is stopped and a sludge skimmer is lowered into the bowl to remove the collected sludge. The feed is then started, initiating another cycle.

Procedures:

The same feed system was used in operating the Fletcher Centrifuge as the P-600 except that no flocculents were added to the Fletcher unit. The feed was started and samples of the centrate were taken on a time basis. As the basket filled with sludge, the centrate suspended solids also increased. At this point the feed was stopped and the sludge skimmed from the basket. The cycle was then repeated.

Results:

Figure 113 presents the results of the Fletcher unit with the primary sludge and the P-600 effluent from a combined sludge run. As the feed rate was decreased, the cycle time and percent recovery increased as might be expected.

Sharples DHL Nozljector

The DHL Nozljector had a recycle clarifier bowl assembly equipped with .050 inch nozzles. With the recycle bowl assembly, it was possible to vary the underflow or cake concentration by varying the feed rate and recycle rate.

Figure 108

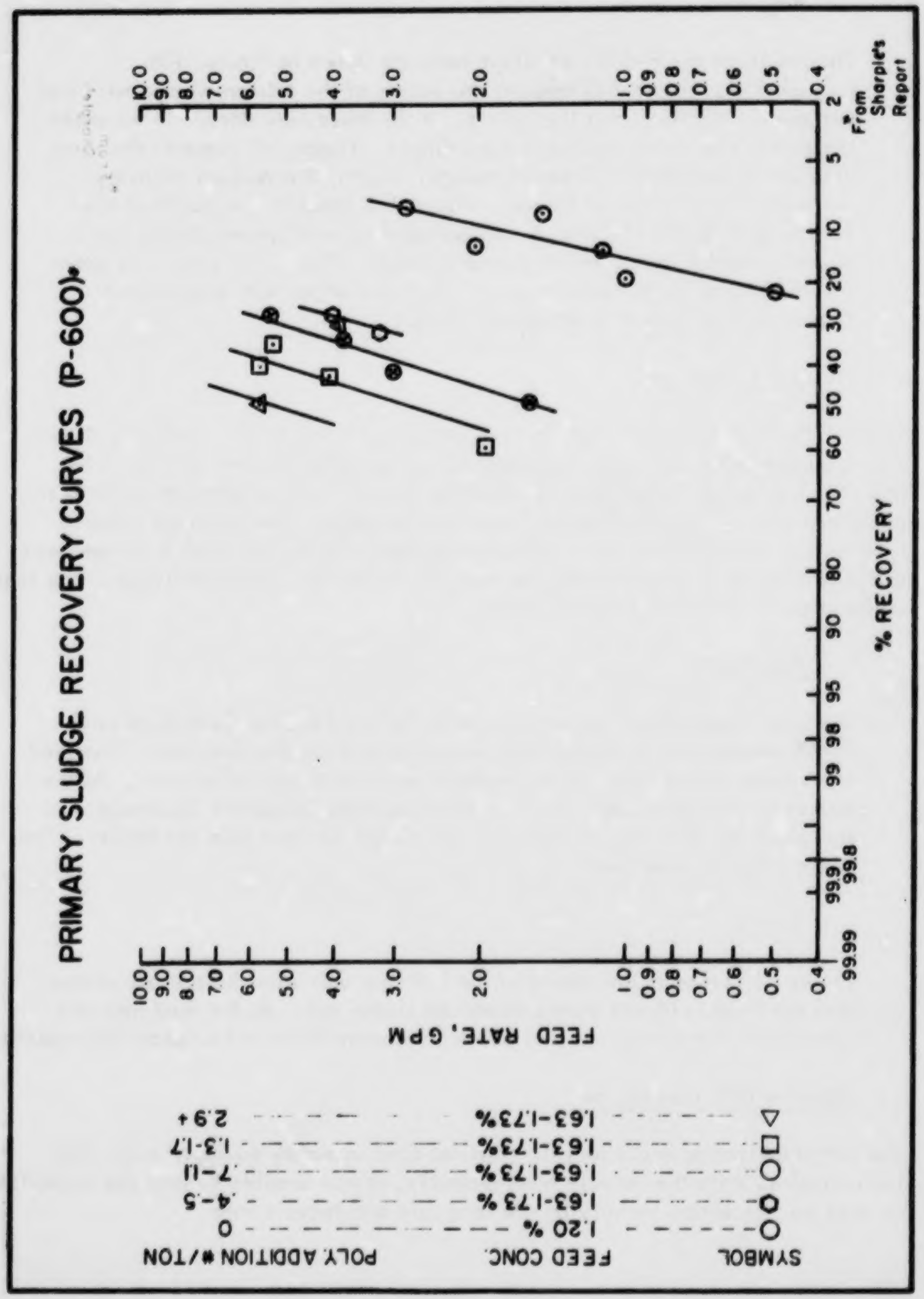
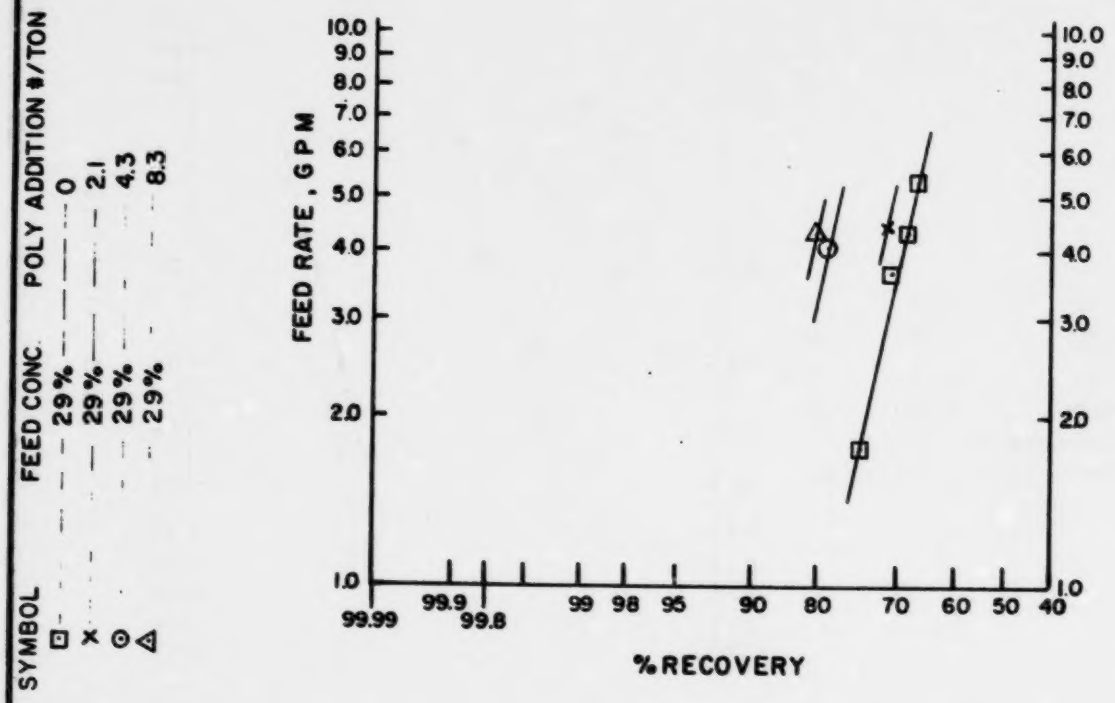

PRIMARY SLUDGE RECOVERY CURVES (P-600)*

Figure 109

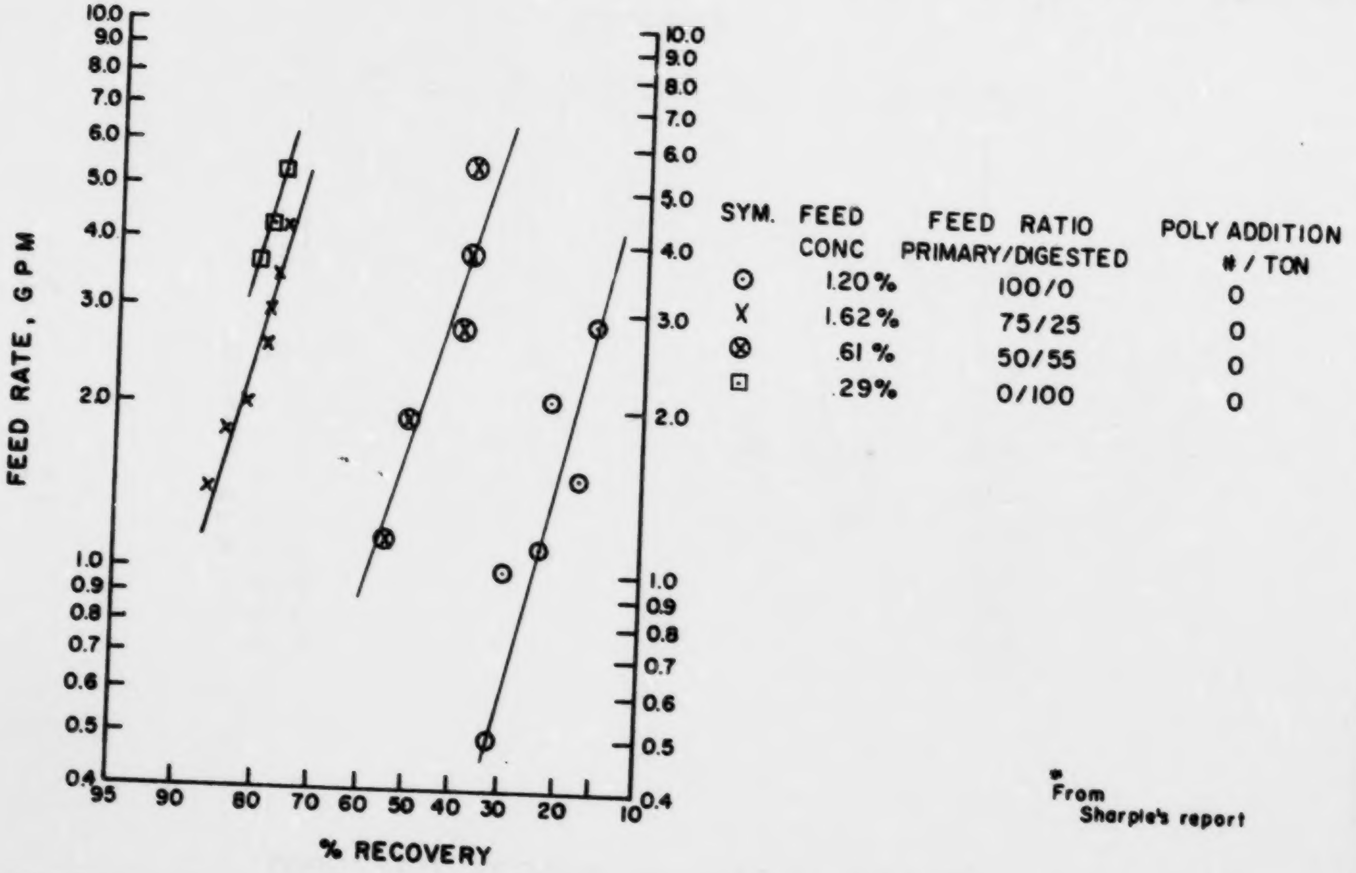
DIGESTED SLUDGE RECOVERY CURVES (P-600)*



*From Sharple's report

Figure 110

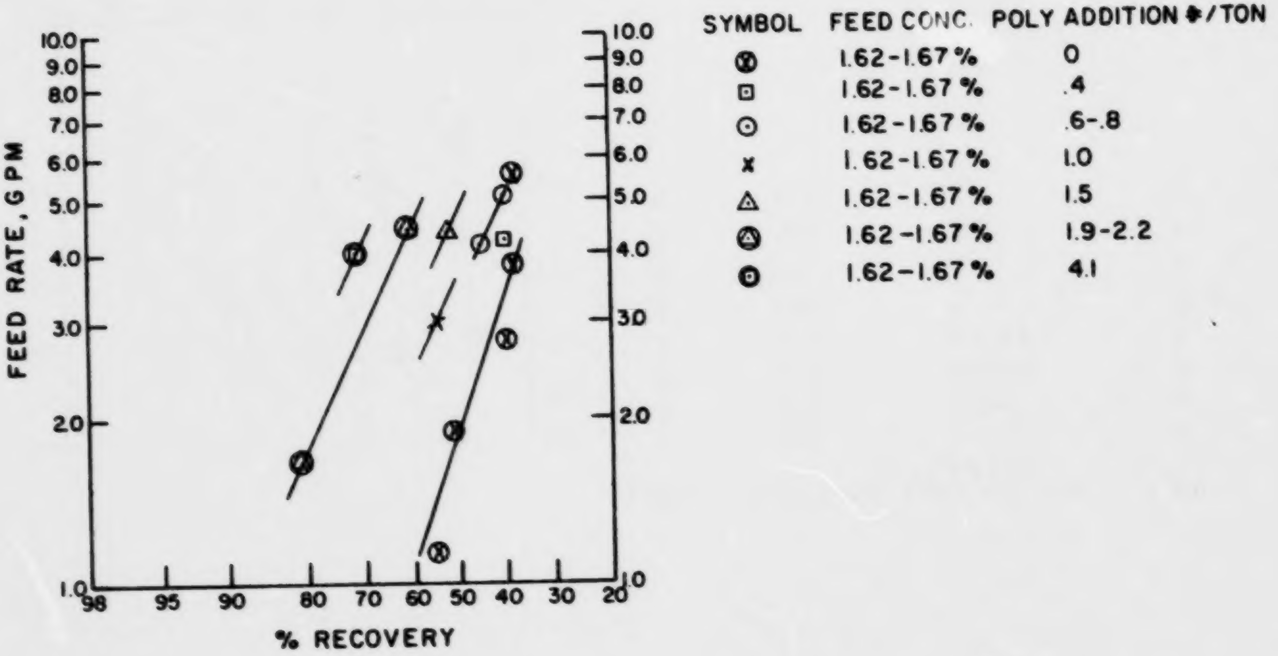
COMBINED SLUDGE RECOVERY CURVES (P-600)*



*From Sharple's report

Figure 111

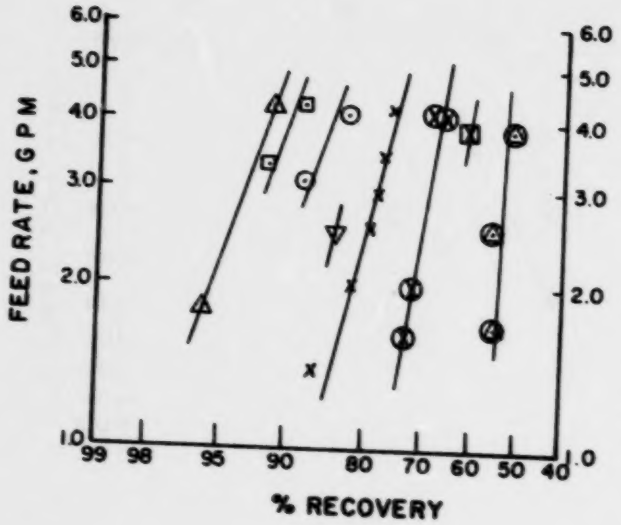
COMBINED SLUDGE RECOVERY CURVES (P-600).



From
Sharple's report

Figure 112

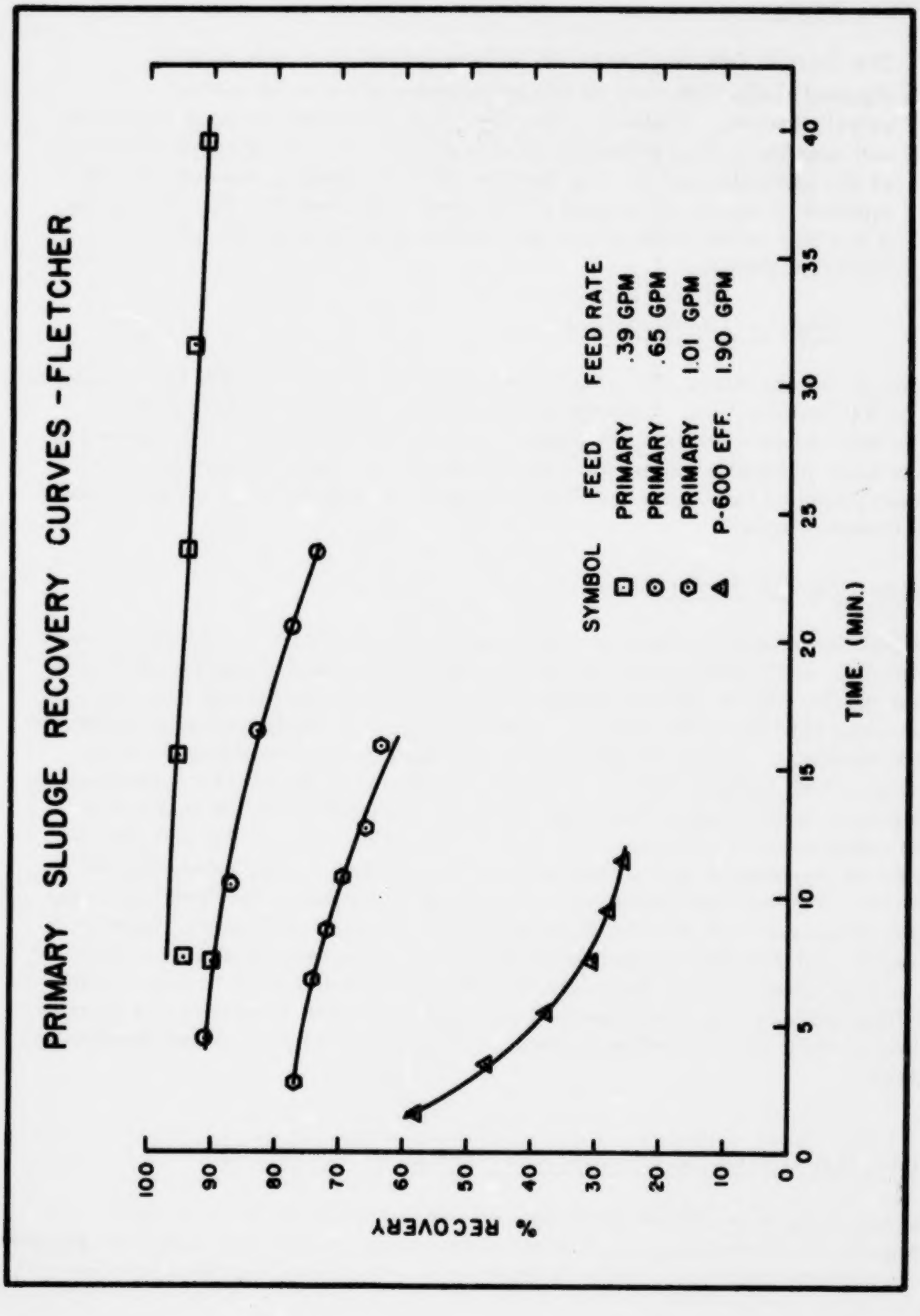
COMBINED SLUDGE RECOVERY CURVES - (P-600)*



SYMBOL	FEED CONC.	POLY ADDITION #/TON	POND
×	.61%	0	4
⊙	.61%	1.1-1.4	4
⊡	.61%	2.0-2.6	4
△	.61%	4.1-4.8	4
⊗	.51%	0	3 1/2
(△)	.51%	0	3
⊡	.51%	2.7	3
▽	.51%	4.2	3

*From Sharple's report

Figure 113



PRIMARY SLUDGE RECOVERY CURVES - FLETCHER

Results:

The Sharples DHL Nozljector was able to process secondary aerobic digested sludge with over 90 percent recovery of the solids without polyelectrolytes. However, installation limitations such as pump capacities and quantity of feed material available did not allow for complete evaluation of this particular model. The Sharples DH-5 Nozljector, however, has been reported to recover 80 percent of the solids in the feed at a flow rate of 200 gpm with a solids increase from one percent to six percent dry solids for municipal sludges.

Summary (Centrifugation)

Based on the centrifuge pilot program previously described, Sharples has recommended a P-5400 Sharples Super-D-Canter with operating curves for a 75/25 primary-secondary sludge ratio shown in Figure 114 and a 50/50 ratio shown in Figure 115. They have included a process requirement summary as shown in Table 43. The values presented here are indicative of the centrifuge performance using Sharples equipment or equal.

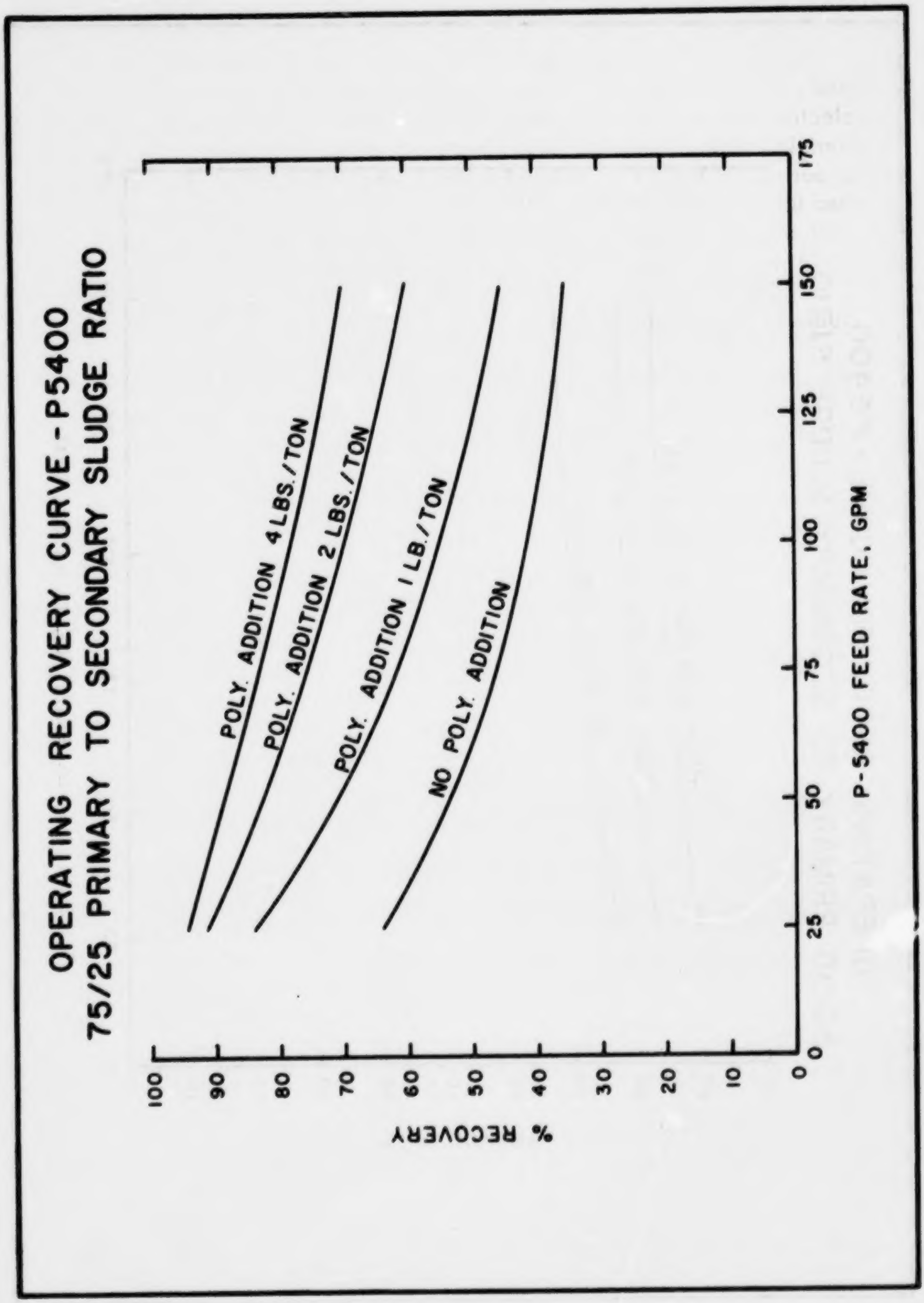
Summary (Sludge Handling)

An analysis of aerobic digestion and sludge dewatering by Filter Pressing, Vacuum Filtration, and Centrifugation has been performed. Aerobic disgestion appears to be a feasible way of reducing approximately half of the VSS wasted from the secondary clarifier to the digestor. The digested solids should then be combined with the primary sludge, thickened, and conveyed to the dewatering facilities. The pilot tests indicate that the combined sludge can be thickened to approximately 38 percent solids using a filter press, although the conditioners are included in this concentration. Influent solids were 1.5 percent, although this concentration might be increased to two to three percent in practice, thereby enhancing the process. The leaf tests indicated vacuum filters can dewater the combined sludge up to a concentration of 20 to 25 percent solids without conditioners. Centrifugation will dewater the combined sludge to approximately 12 percent with or without conditioners. The results of these studies would favor vacuum filtration or filter pressing over centrifugation, although the process economics and ultimate disposal of the sludge itself will strongly influence the selection of the dewatering system.

PILOT PLANT PROCESS EVALUATION - EFFLUENT POLISHING

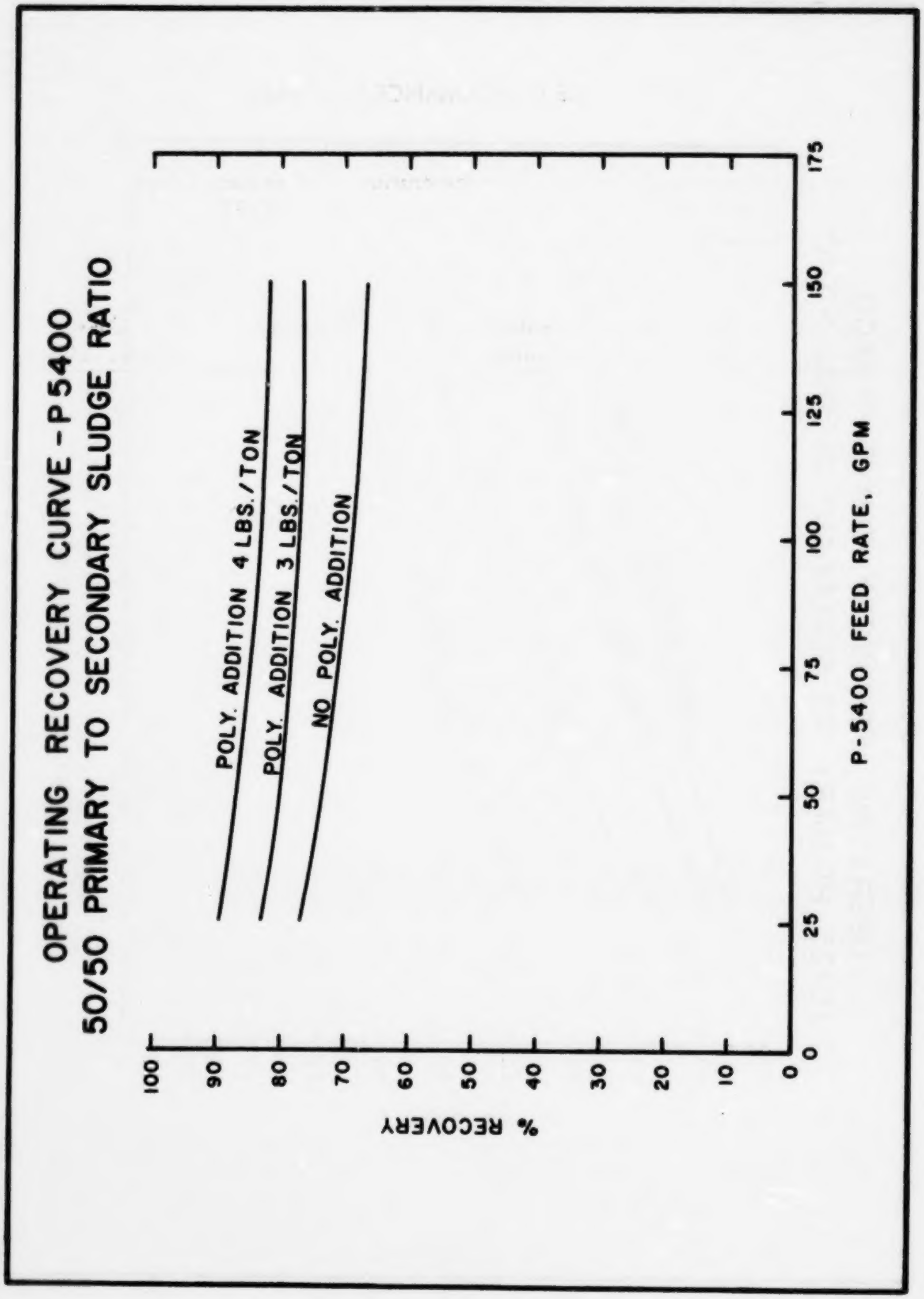
Several methods of effluent polishing were evaluated during the pilot plant program. An extensive evaluation of carbon adsorption was made using both granular

Figure 114



OPERATING RECOVERY CURVE - P5400
75/25 PRIMARY TO SECONDARY SLUDGE RATIO

Figure 115



OPERATING RECOVERY CURVE - P 5400
50/50 PRIMARY TO SECONDARY SLUDGE RATIO

TABLE 43

CENTRIFUGE PERFORMANCE SUMMARY

Conditions: Influent suspended solids concentration: 1.6 percent solids
Approximate primary/secondary solids ratio: 75/25
Equipment: Sharples P-5400

Feed Rate GPM	Polyelectrolyte Addition lbs Poly/ton feed solids	Recovery %	Cake % total solids
50	0	54	12
50	2	83	12
50	4	90	12
100	0	42	13
100	2	70	12
100	4	78	12
150	0	35	13
150	2	60	12
150	4	68	12

and powdered activated carbon. Additionally, effluent sand filtration and microstraining were evaluated. This section describes the procedures followed during these tests and presents the results as related to design criteria.

Activated Carbon Adsorption Evaluation

In general, two types of experimental procedures were utilized for an evaluation of activated carbon adsorption as a method of wastewater treatment, namely adsorption isotherms and adsorption column studies. The amount of substance adsorbed per unit weight of carbon can be investigated by the preparation of adsorption isotherms. Isotherms can also be used to develop a general estimate of carbon column efficiency, though caution must be exercised as other removal phenomena occur in actual column operations. The prime advantage of adsorption isotherm studies is that they can be performed on a batch basis and thus provide a rapid method for screening the relative efficiencies of various carbon types and the susceptability of a given wastewater to treatment.

Conversely, adsorption column studies require considerable equipment and extended periods of operation for the development of meaningful data. Column studies are however, the best available method for developing design criteria for a specific wastewater.

Adsorption Isotherm

A series of adsorption isotherm experiments were performed to investigate the feasibility of carbon adsorption as a method of treatment. Additionally, this method was utilized to screen several types of commercial carbon to determine which was the most effective. Tests were performed on untreated wastewater, wastewater after neutralization and primary settling and effluent from the pilot biological treatment unit.

Adsorption isotherm tests were performed by mixing predetermined amounts of activated carbon with a solution of known contaminant concentration. The batch system was then mixed until adsorption equilibrium had been reached after which the final concentration of the contaminant in solution was determined.

When this procedure is followed for a given wastewater using several different carbon dosages, the results will generally conform to the Freundlich isotherm, described by Equation VI-14,

$$x/m = kc^{1/n} \qquad \text{(VI-14)}$$

where x/m is the carbon loading in lb. of contaminant per
lb. carbon, c is the equilibrium concentration and
k and n are constants.

If plotted on log-log paper, the data normally defines a straight line which is representative of the capacity of the carbon to adsorb a given contaminant from the wastewater for a given initial concentration. Since powdered activated carbon is generally mixed with the wastewater to be treated in precisely the same manner as the test procedure, the adsorption isotherm gives a direct measurement of the carbon dosage required to reach a given purity level. However, in the application of granular activated carbon in columns, other removal mechanisms occurred and isotherm studies can provide only a generalized estimate of the results to be expected.

Normally, powdered activated carbon is used to perform isotherm studies because equilibrium is attained more rapidly and reliable results can be obtained within 30 minutes of contact. However, isotherms developed using powdered carbon are not always representative of what would occur using granular material. Because granular carbon exhibits a much lower adsorption rate than the powdered material, a sufficient contact time must be allowed.

Tests were performed to determine the contact time required for several types of granular activated carbon to reach adsorption equilibrium in samples of untreated, primary and secondary effluents. In all cases equilibrium occurred within three hours of contact.

Based upon the results of these studies, isotherms were performed using raw wastewater, primary effluent and secondary effluent with three brands of activated carbon and allowing three hours for equilibrium to be obtained. The adsorption isotherms of powdered carbons were also determined. Performance was measured in terms of COD and color as determined by platinum-cobalt standards.

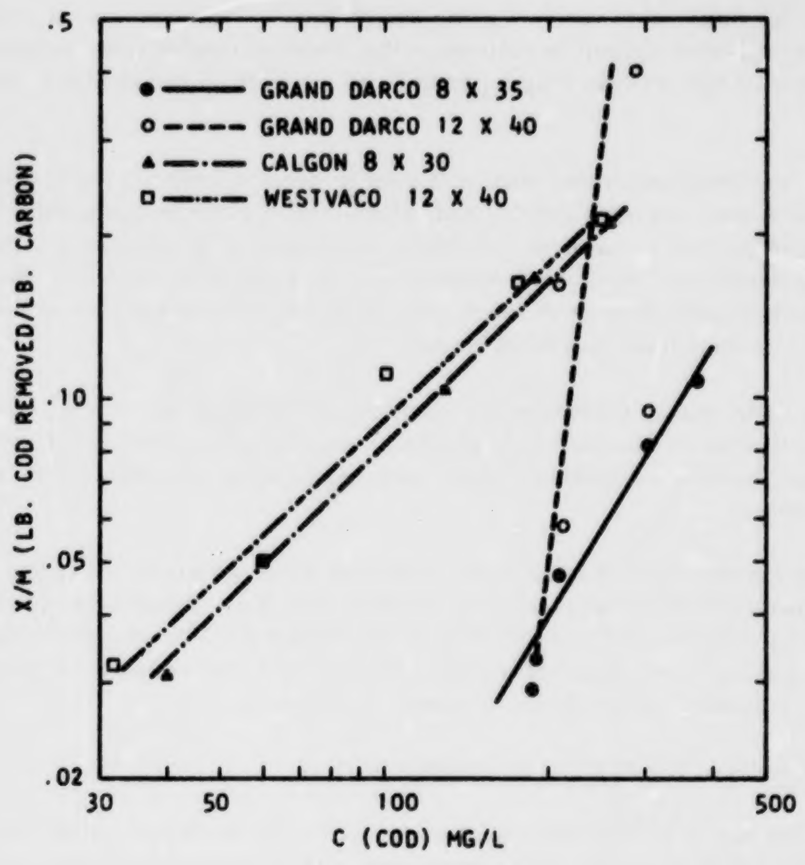
Results of the Adsorption Isotherm Studies:

The results of the batch adsorption studies are presented graphically in Figures 116 through 121, with associated carbon capacity estimates summarized in Table 44. Plotting of the batch adsorption data in the Freundlich isotherm format allows the rapid estimation of carbon capacity at exhaustion for a given influent concentration of contaminant. Perhaps the most pertinent development of the batch adsorption studies is that the resulting estimates of adsorptive capacity generally fall in the range indicative of economically feasible activated carbon treatment.

This conclusion was reached by virtue of the fact that existing carbon

Figure 116

ADSORPTION ISOTHERM - COD UNTREATED WASTEWATER



ADSORPTION ISOTHERM - COD PRIMARY TREATMENT EFFLUENT

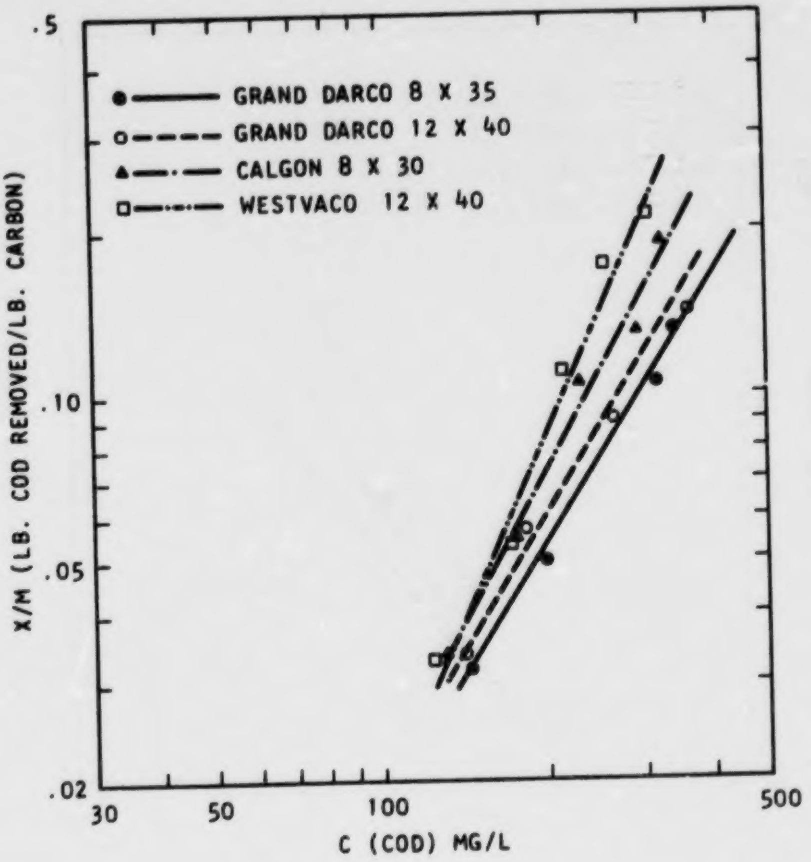


Figure 118

ADSORPTION ISOTHERM - COD BIOLOGICAL TREATMENT EFFLUENT

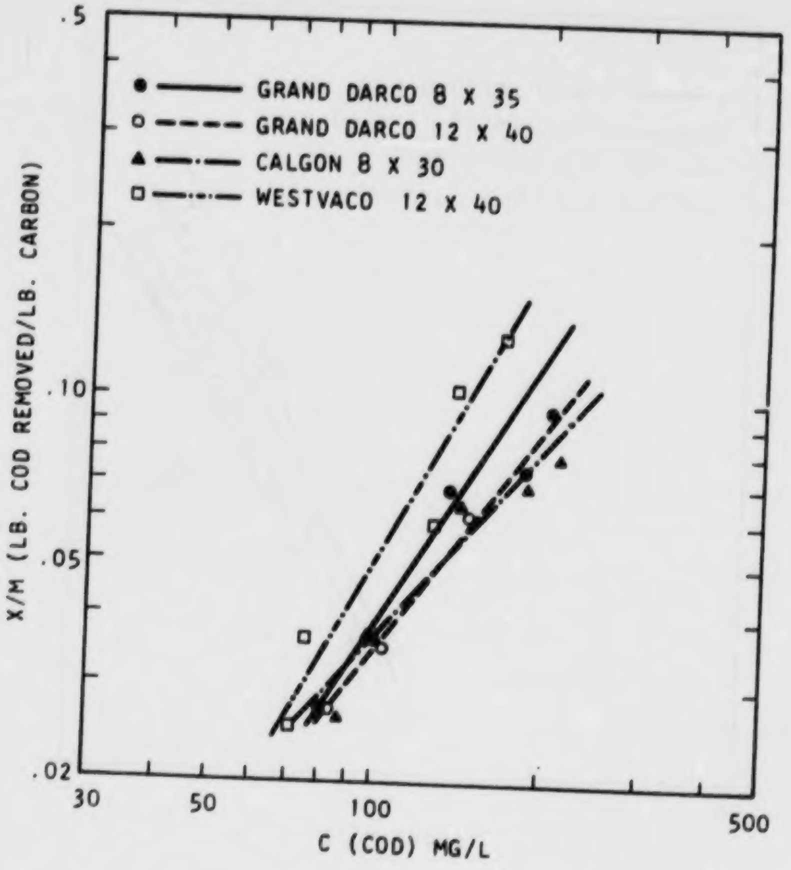


Figure 119

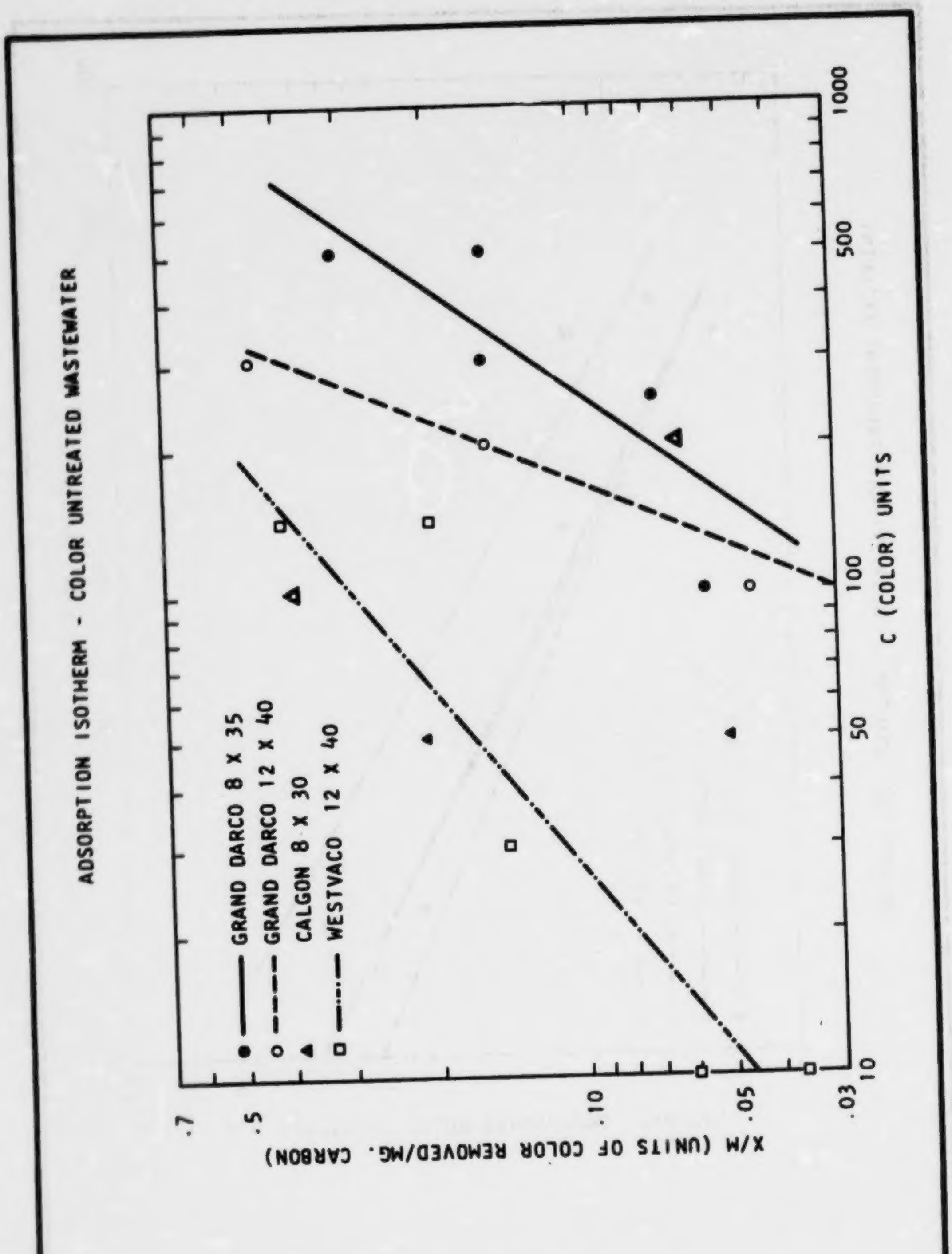
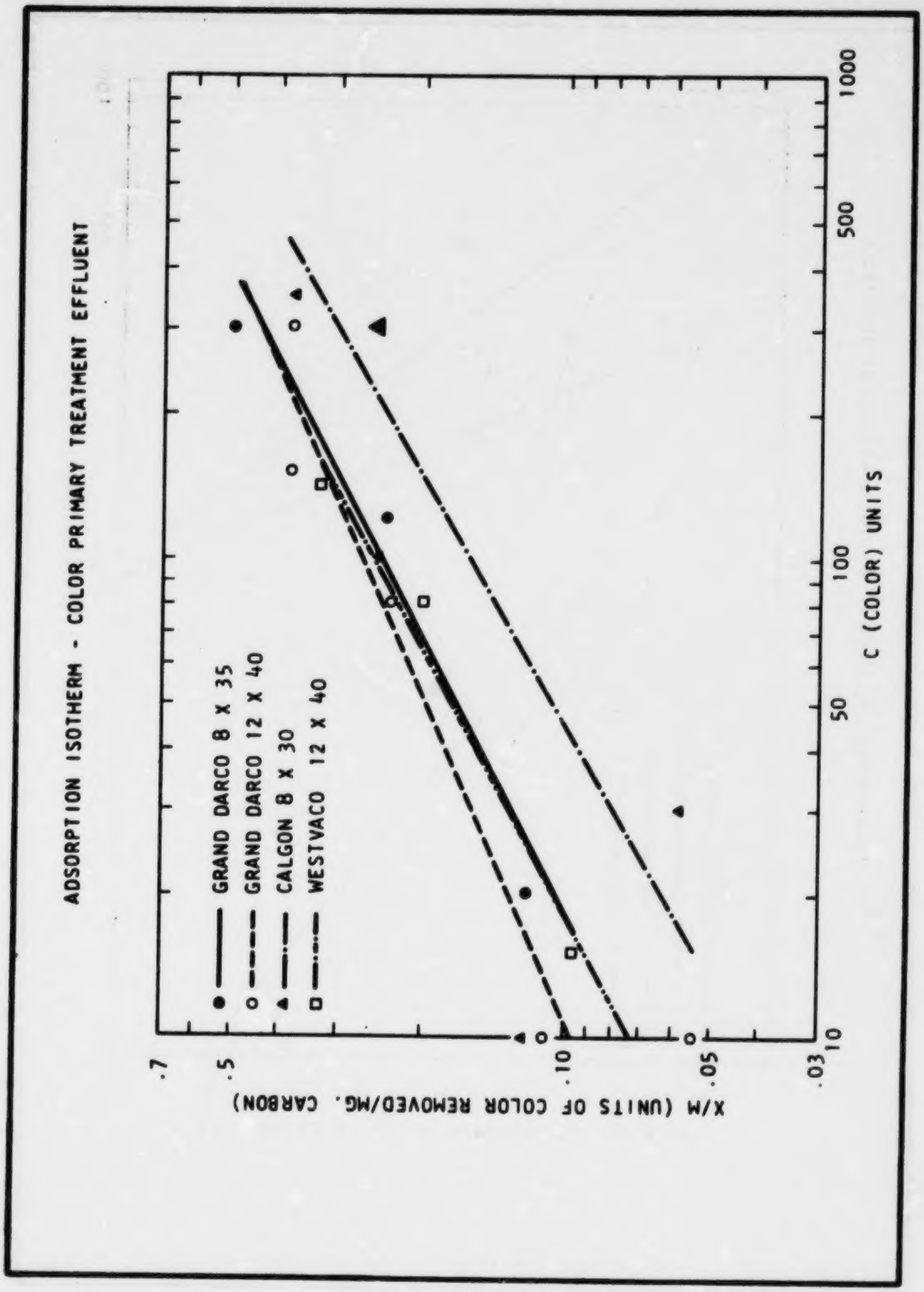
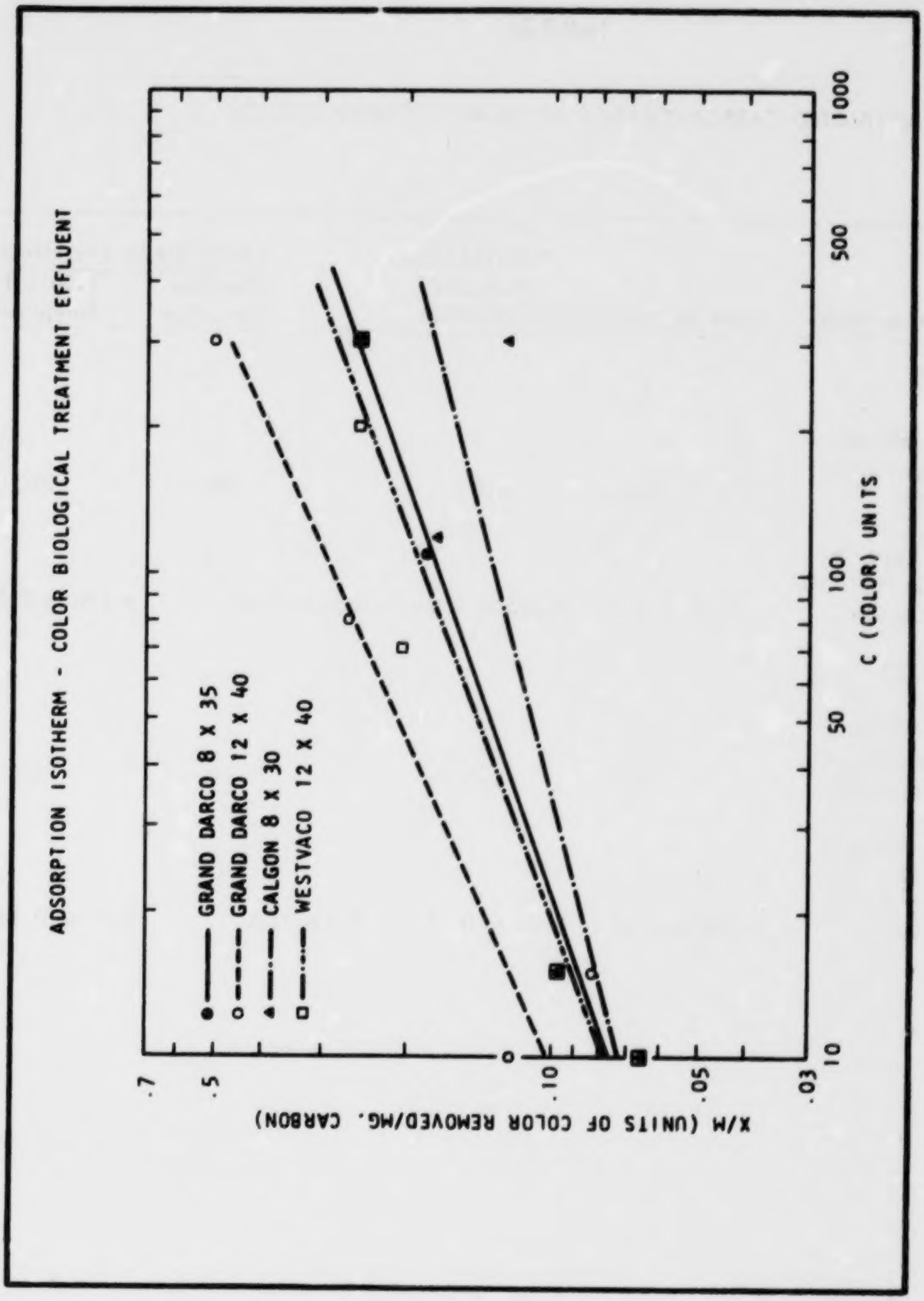

ADSORPTION ISOTHERM - COLOR UNTREATED WASTEWATER

Figure 120



ADSORPTION ISOTHERM - COLOR PRIMARY TREATMENT EFFLUENT

Figure 121



ADSORPTION ISOTHERM - COLOR BIOLOGICAL TREATMENT EFFLUENT

TABLE 44

ACTIVATED CARBON CAPACITIES FROM ISOTHERM STUDIES

			SECONDARY EFFLUENT	
PARAMETER	RAW WASTEWATER	NEUTRALIZED PRIMARY EFFLUENT	Six-Hour Detention	Twelve-hour Detention
COD				
(1) Influent concentration (mg/l)	530	410	320	250
(2) Capacity Range lbs COD/ lbs Carbon	0.20 to 0.45	0.175 to 0.440	0.26 to 0.42	0.170 to 0.275
COLOR				
(1) Influent concentration (color units)	700	550	500	500
(2) Capacity Range units/mg carbon	0.39 to 2.80	0.43 to 0.65	0.2 to 0.6	0.17 to 0.44

treatment facilities operate in the range of 0.25 to 0.5 pounds of COD removed per pound of carbon regenerated. Experience has indicated that carbon utilization in full scale facilities is 50 to 100 percent more efficient than was predicted from adsorption isotherms. Even without this 50 to 100 percent surcharge, the carbon capacity estimates shown in Table 44 fall within the accepted range of economic feasibility. Considering Table 44, increasing the degree of pre-treatment had little effect upon carbon capacity, except in the case of color removal, where neutralization and primary settling actually appeared to increase adsorption capacity. The apparent decrease in capacity experienced when biological treatment was extended from six to twelve hours can be attributed to a reduction in assumed influent concentration, rather than a significant change in adsorptive capability. In all cases, the isotherm indicated that the organic contaminants responsible for color in the wastewater are selectively adsorbed. Therefore, color removal should be relatively more efficient than the removal of the entire spectrum of organic contaminants as reflected by COD. As expected, the smaller particle size of the 12 x 40 mesh granular carbon exhibits the greatest capacity for both COD and color due to its larger surface area.

The granular carbon produced by the Westvaco Corporation consistently exhibited superior capacity for COD, whereas, Grand Darco carbon manufactured by the Atlas Chemical Company was superior in color removal capabilities. Based upon these results, Westvaco 12 x 40 carbon was selected for bench scale column studies. However, other factors, such as a chemical resistance and durability must be considered for the final selection of granular carbon for full scale facilities.

Bench Scale Carbon Column Studies

A series of four carbon column experiments were performed to further evaluate the feasibility of carbon sorption as a treatment process. Additionally, data was gathered to develop design criteria for cost analysis purposes. Three of the experiments utilized the down flow packed column mode of contact, two of these being performed upon effluent from the pilot biological treatment plant in order to evaluate activated carbon in a purely tertiary treatment role. The third down flow experiment was conducted upon wastewater that had received neutralization and primary sedimentation. Another study was performed using the upflow expanded bed mode of contact, the influent to the columns being raw wastewater.

Six 2.9 inch I. D. Plexiglas columns six feet in length and associated stainless steel tubing and valving composed the major elements of testing equipment. Prior to beginning an experiment, each column was loaded with five pounds of activated carbon to an average depth of 44 inches. Flow through the columns was provided by a small variable speed centrifugal pump, with flow rate measurement accomplished by a rotameter. Valves were strategically placed in the piping system so that individual columns could be backwashed at essentially any desired flow rate using the variable speed pump. Effluent from the final column was collected and stored for backwashing. In the down flow mode of contact, the first column was backwashed weekly as dictated by head loss.

During the course of experiments using raw wastewater as the influent, a slip stream from the equalization tank discharge line was routed to the surge tank adjacent to the carbon columns. The column feed pump then took suction from the surge tank. On subsequent experiments, primary and secondary effluents were siphoned from the primary and secondary clarifiers for discharge to the surge tank. The continuous column test apparatus is shown in Figure 122.

Sampling and Analysis Schedule:

Influent to the columns and effluent from the final carbon columns were sampled twice daily. Influent grab samples were taken from the surge tank, whereas effluent from the final column was stored under refrigeration and the resulting composite sampled. The effluent from intermediate columns was sampled on a daily basis. The volume of through-put was recorded twice daily in conjunction with the sampling effort.

Chemical oxygen demand was the only parameter investigated during experiments using effluent from the primary clarifier. However, a much more inclusive analysis schedule was followed for the other experiments. The schedule included analysis for the following parameters:

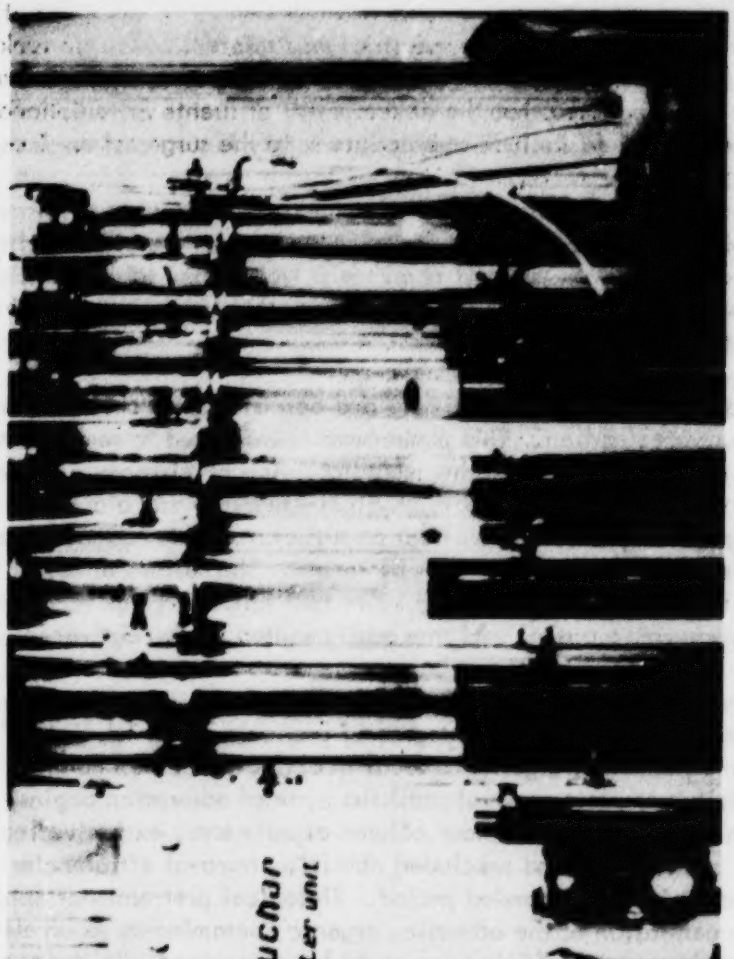
a. Chemical Oxygen Demand
b. Total Organic Carbon
c. Total Oxygen Demand
d. Biochemical Oxygen Demand
e. Phenolic materials
f. Total Kjeldahl Nitrogen
g. Total Nitrates
h. Total Phosphates
i. Color (Spectrophotometric)

Discussion of Results:

To increase clarity and reduce the volume of tabular data, the results of the column studies are presented in graphical form wherever possible. In many cases, these graphs depart significantly from the clear cut geometry

Figure 122

CARBON COLUMN TESTING APPARATUS


nuchar
filter unit

expected from theoretical concepts. These departures from normally accepted form can be attributable to the following:

a) influent concentrations significantly higher than normal municipal wastewaters;

b) continually changing influent concentrations and characteristics; and

c) the complex makeup industrial wastewater.

It must be recognized that the development of design criteria from such studies by necessity should include appropriate safety factors and engineering judgment.

It is recognized that the carbon column experiments were performed on a wastewater at different stages of pretreatment, and varying results would be expected. However, several phenomena were found to occur irrespective of the degree of pretreatment, and therefore, this can be attributed to fundamental characteristics of the wastewater.

The first of these involves the ease and consistency of color removal by treatment with activated carbon. This phenomena is indicated to some extent by the results of the adsorption isotherm studies. The consistency of removal can be explained by the fact that both physical-chemical and biological pretreatment steps apparently have little effect on either true color concentration or the nature of the substance responsible for coloration. The failure of biological treatment to significantly reduce color indicates that large complex organic molecules are the causitive agent, and thus easily sorbed by the activated carbon.

Another interesting phenomena is the excessive leakage of certain organic contaminants irrespective of degree of pretreatment or loading. The normally expected pattern of organic removal in activated carbon columns entails essentially complete removal until the zone of adsorption begins to exit the column. However, in all four column experiments, excessive leakage began almost immediately and precluded obtaining removal efficiencies exceeding 90 percent for any extended period. Biological pretreatment apparently reduced the concentration of the offending organic contaminants to levels where acceptable removal efficiencies could be maintained. In the experiments involving raw wastewater and primary effluent, leakage of adsorption resistant compounds increased with loading producing what appeared to be an initial break-through. Quite possibly, the leakage is composed predominantly of low molecular weight organic compounds susceptible to biological removal but highly resistant to adsorption by activated carbon.

As expected, the removal of the primary nutrients, nitrogen and phosphorous, by carbon sorption was unimpressive. This effect was typified by the removals experienced in treating the secondary biological effluent. Apparently, only that portion of nutrients bound up in adsorbable organic molecules can be effectively removed. Conversely, the removal of phenol was highly efficient with effluent concentration never exceeding 0.1 mg/l. These characteristics are depicted in Figures 123 through 126.

Results of Carbon Adsorption of Untreated Wastewaters

Carbon adsorption studies were conducted on the untreated influent to the pilot plant. The wastewater was serially routed through six adsorption columns each containing approximately five pounds of Westvaco 12 x 40 mesh activated carbon. An expanded bed upflow mode of contact was selected for the tests in order to eliminate plugging problems. A linear flow velocity of 8.07 gpm/ft^2 was maintained, thereby providing a total contact time of approximately 21 minutes and a carbon bed expansion which varied between 20 and 30 percent. Other pertinent operation data are summarized in Table 45.

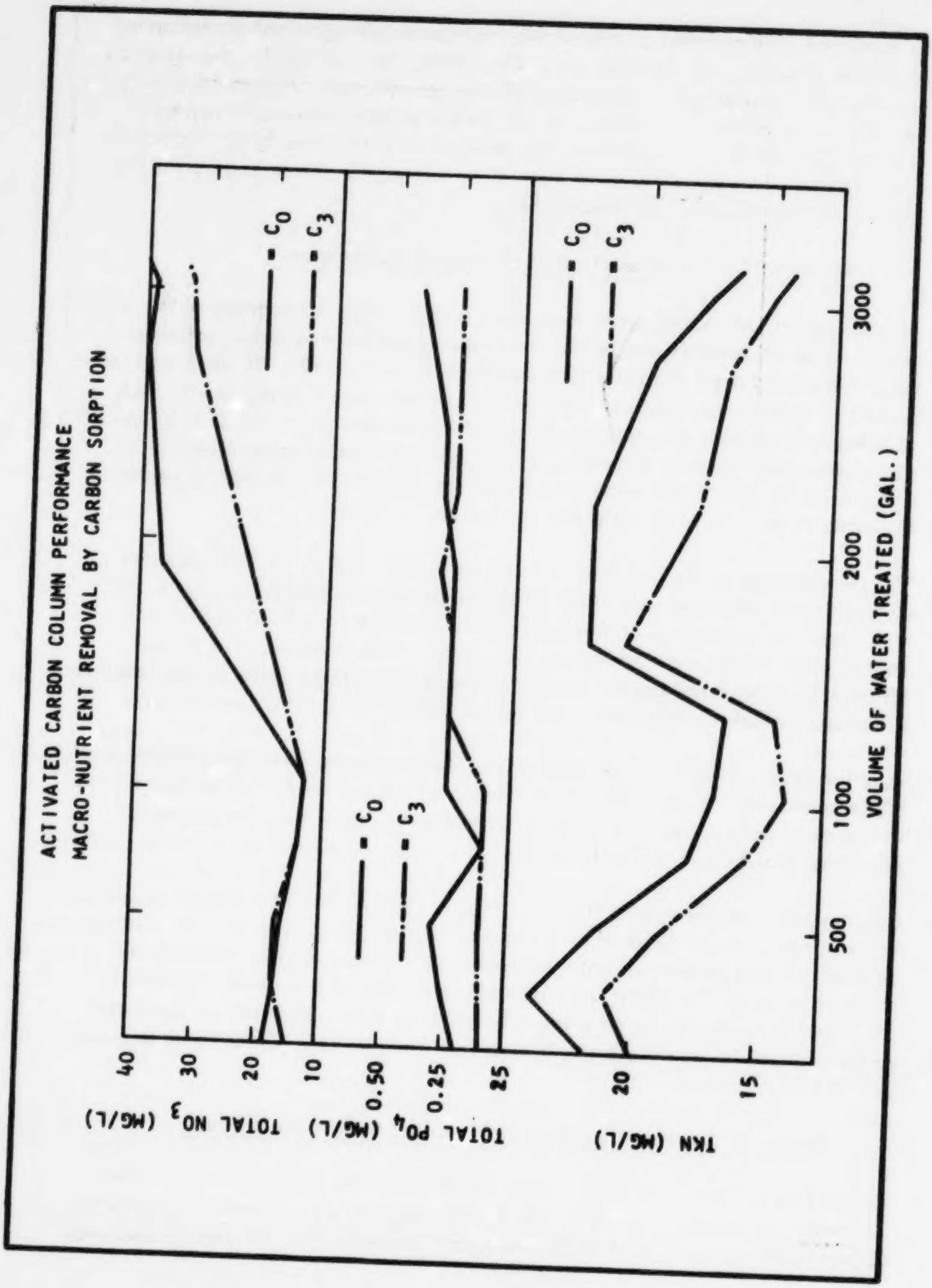
The performance of the columns in terms of BOD, COD, and TOC removal is graphically depicted in Figures 127 through 131. Considering Figure 127, it is apparent that effective treatment is not feasible on a BOD basis due to an excessive leakage of biodegradable organic contaminants. The initial effluent from the final column exceeded projected release criteria and leakage increased linearly to approach influent concentrations. This conclusion is reinforced by the COD and TOC data plotted in Figures 128 and 129. Judging from the BOD_5/COD ratio of the residual contaminants, they are predominantly biodegradeable. Apparently, an extension of column length or contact time would serve only to retard the observed leakage, and within the bounds of economic feasibility, probably would not provide an effective treatment system.

Column performance data is shown in Figure 130 and 131 in a format to reflect percent contaminant removal as a function of the cumulative mass of contaminant applied. These graphs validate the inability of the carbon system to meet removal criteria. In addition to leakage problems, the removal of sorbable COD and TOC proved to be relatively inefficient as indicated by measured carbon capacities at exhaustion of approximately 0.45 lb COD/lb carbon and 0.1 lb TOC/lb carbon.

Results of Carbon Adsorption of Neutralized Primary Effluent

Six packed columns operating in the downflow series mode of contact were utilized to evaluate the affinity of neutralized primary effluent for activated carbon treatment. A total of 30 pounds of Westvaco 12 x 40 mesh carbon was

Figure 123



ACTIVATED CARBON COLUMN PERFORMANCE
MACRO-NUTRIENT REMOVAL BY CARBON SORPTION

Figure 124

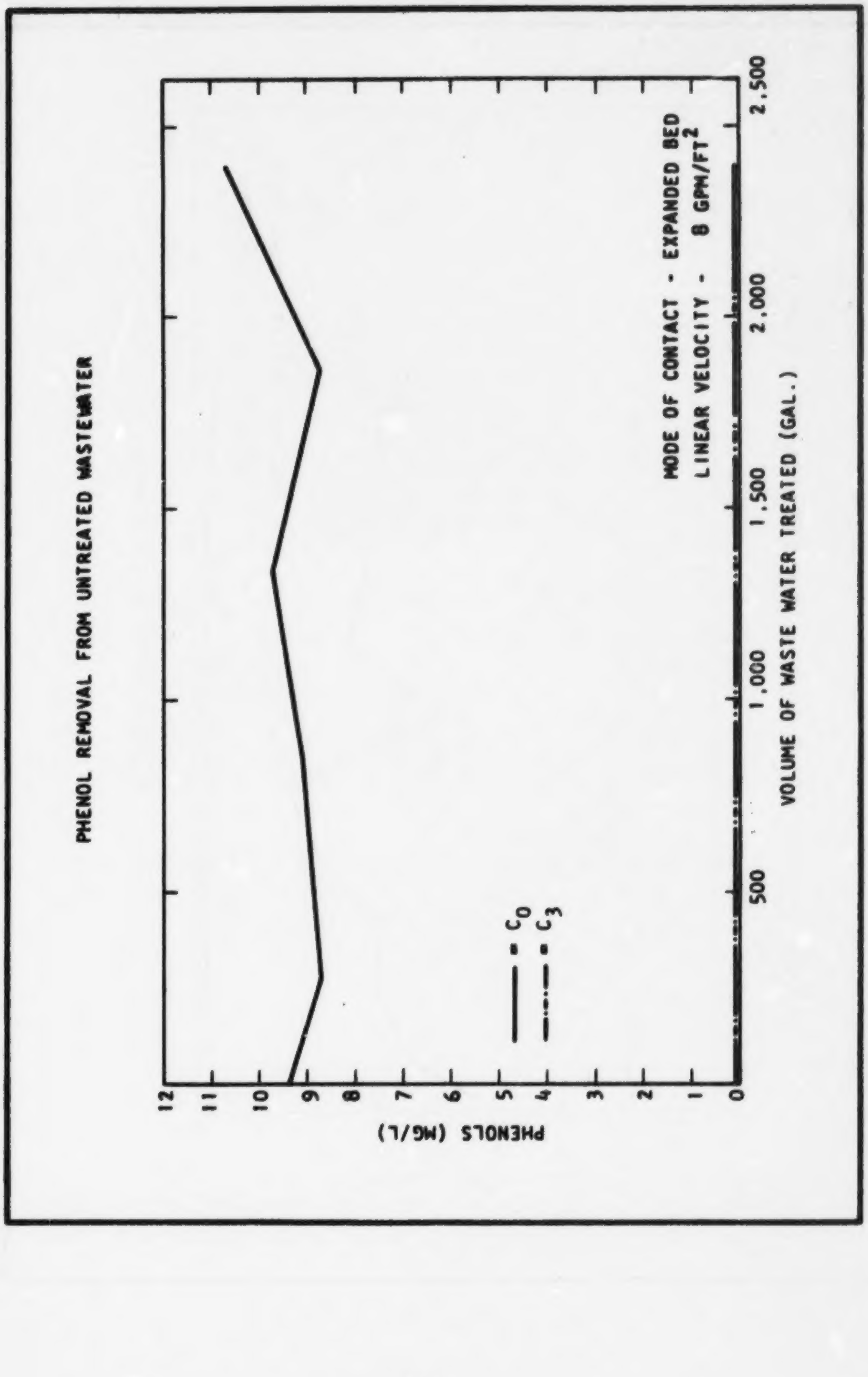

PHENOL REMOVAL FROM UNTREATED WASTEWATER
PHENOLS (MG/L)
12
11
10
9
8
7
6
5
4
3
2
1
0
500
1,000
1,500
2,000
2,500
VOLUME OF WASTE WATER TREATED (GAL.)
= C_0
= C_3
MODE OF CONTACT - EXPANDED BED
LINEAR VELOCITY - 8 GPM/FT^2

Figure 125

PHENOL REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (4.5 GPM/FT^2)

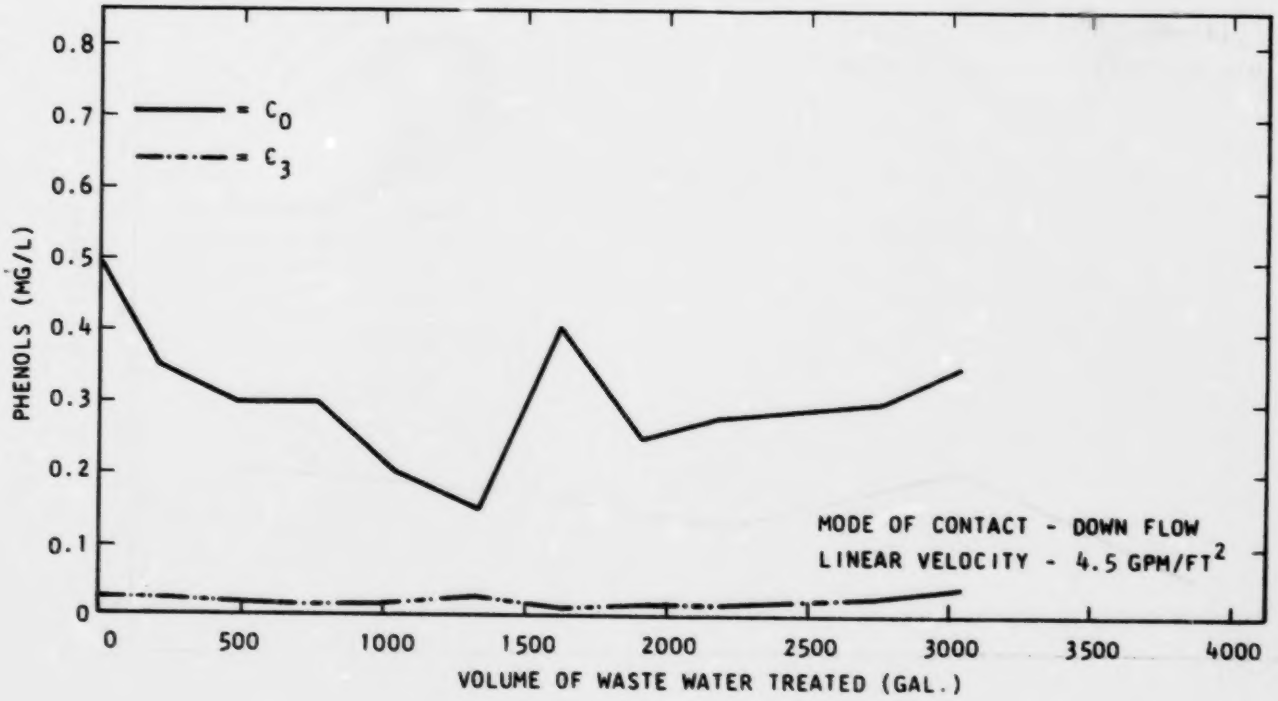


Figure 126

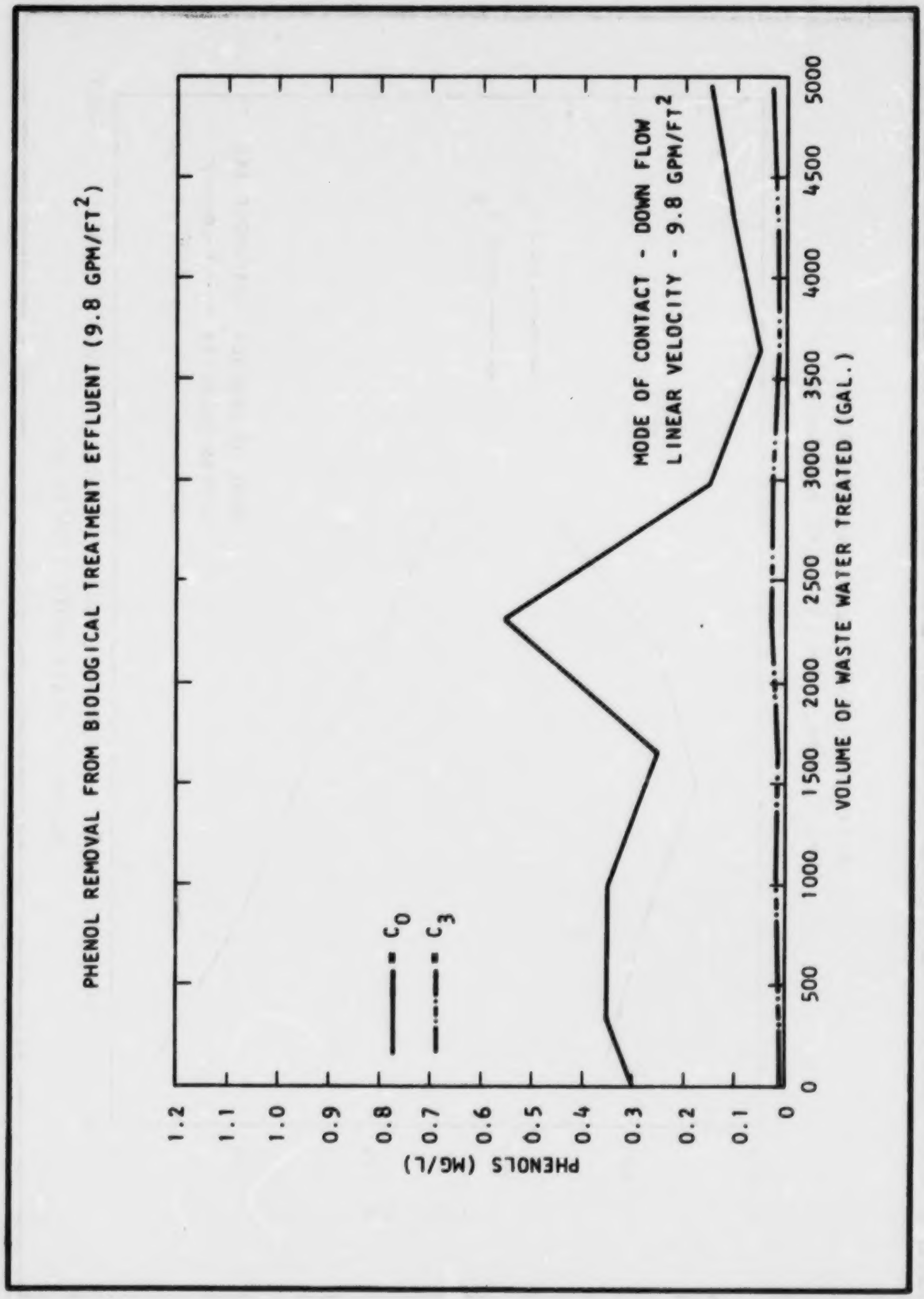
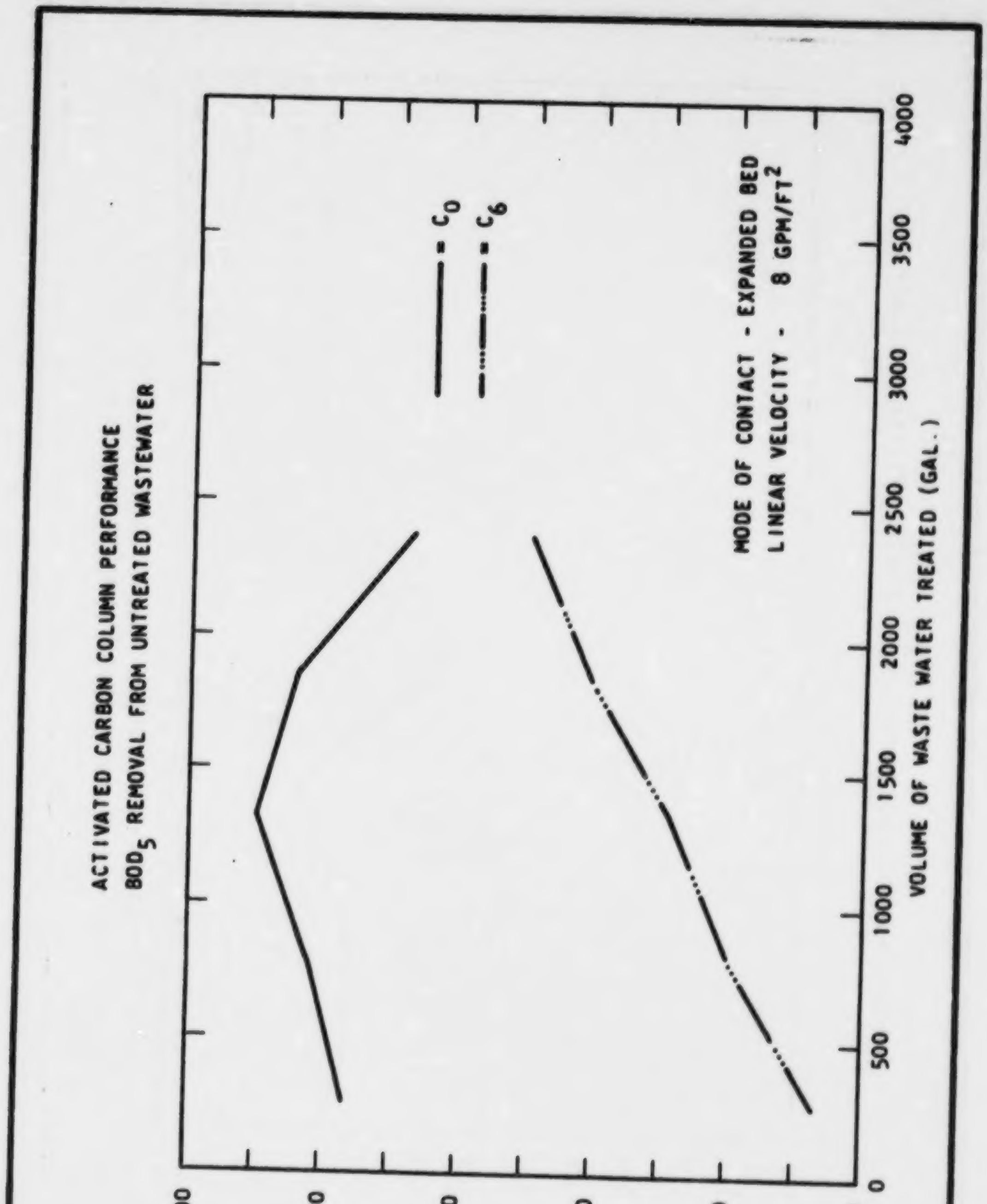
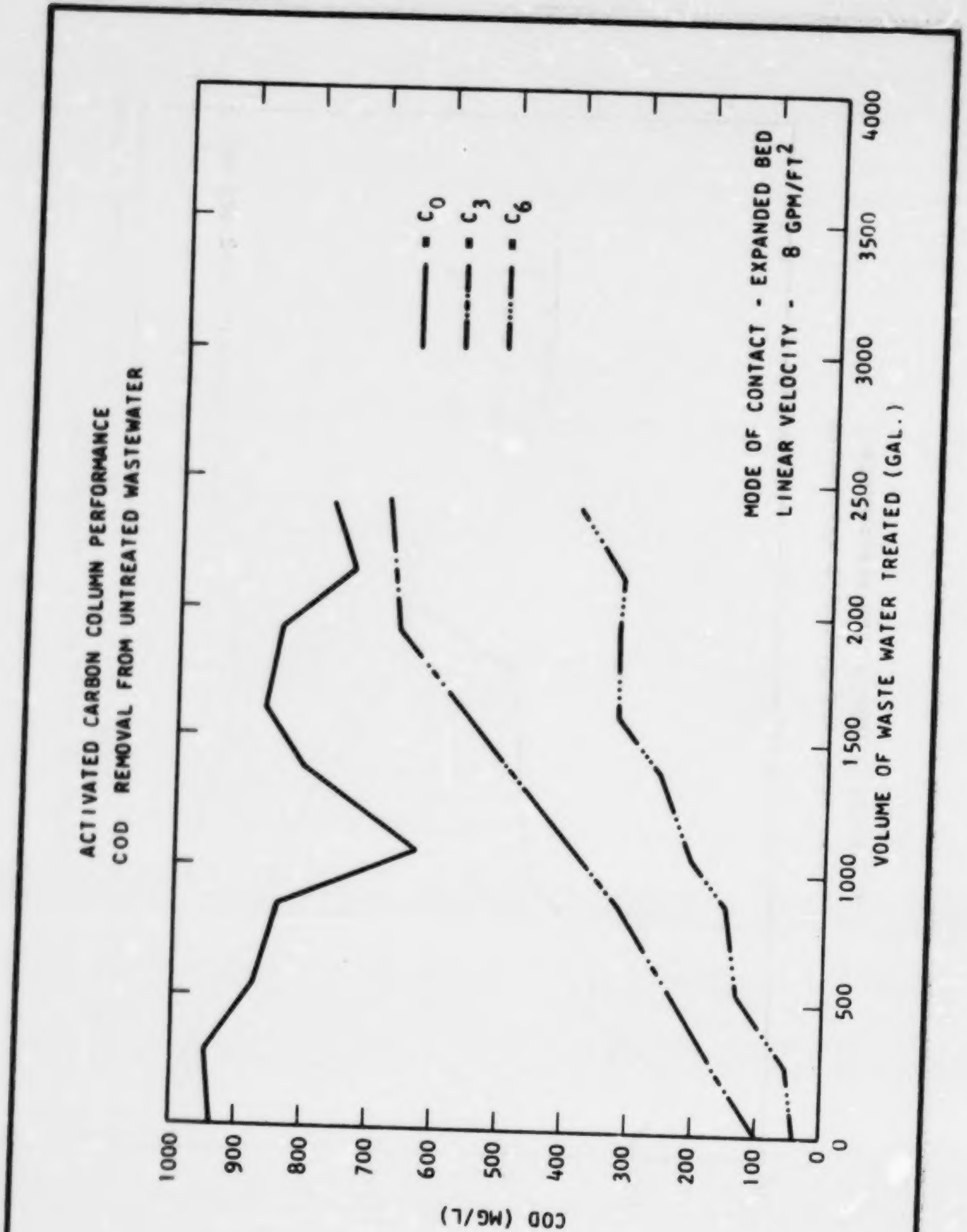

PHENOL REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (9.8 GPM/FT²)

Figure 127



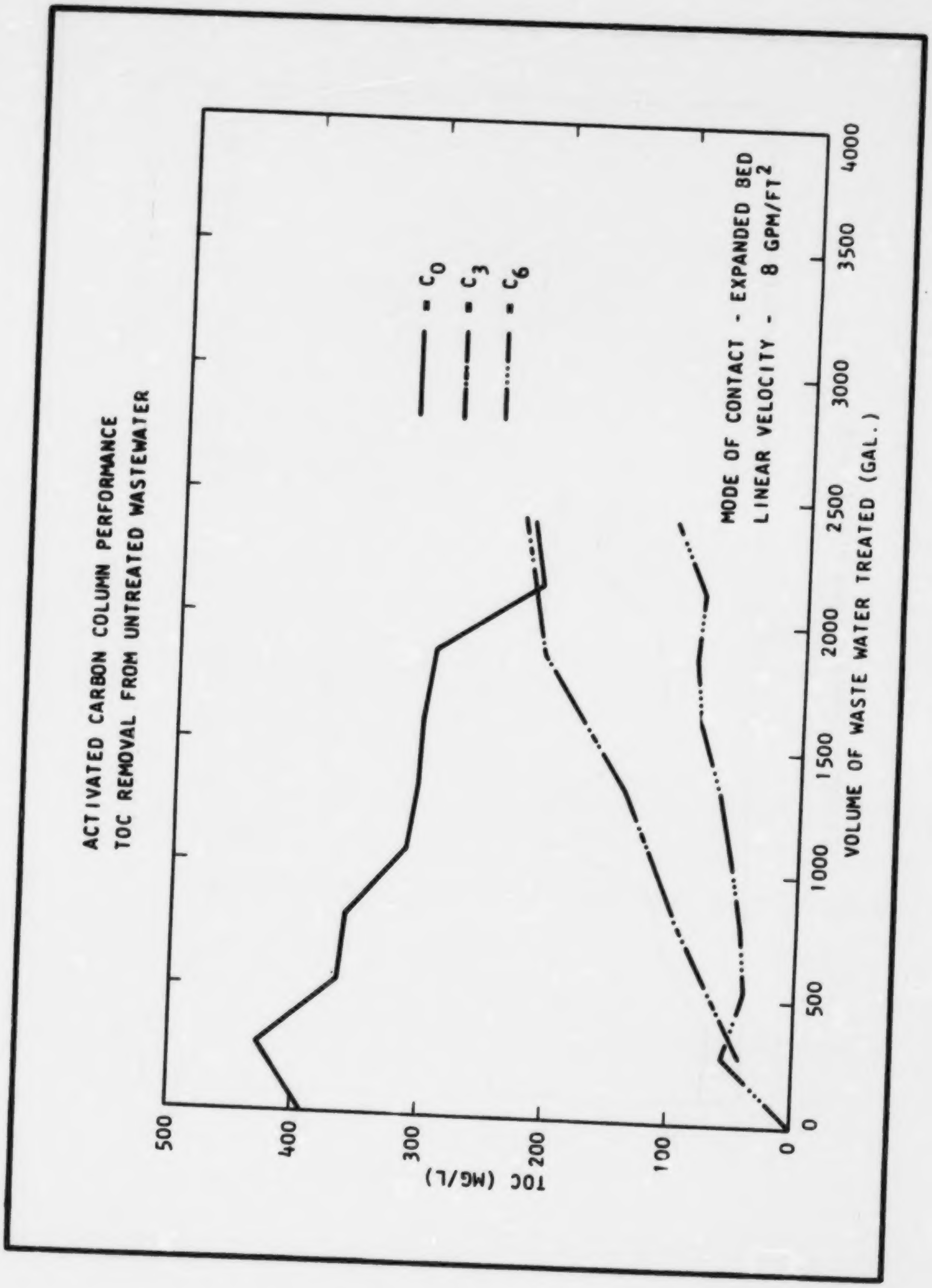
ACTIVATED CARBON COLUMN PERFORMANCE
BOD_5 REMOVAL FROM UNTREATED WASTEWATER

Figure 128

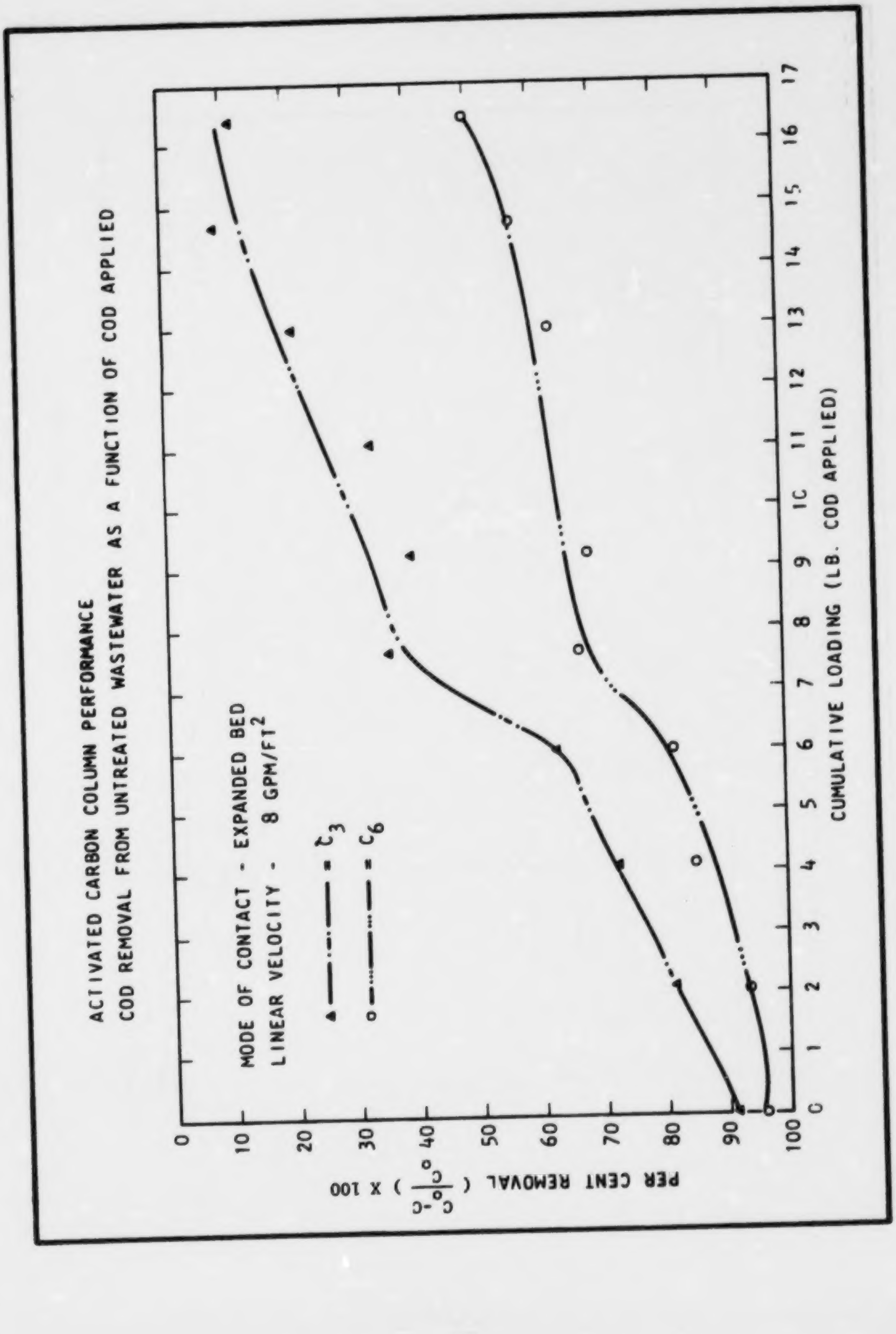


ACTIVATED CARBON COLUMN PERFORMANCE
COD REMOVAL FROM UNTREATED WASTEWATER

Figure 129



ACTIVATED CARBON COLUMN PERFORMANCE
TOC REMOVAL FROM UNTREATED WASTEWATER



ACTIVATED CARBON COLUMN PERFORMANCE
COD REMOVAL FROM UNTREATED WASTEWATER AS A FUNCTION OF COD APPLIED

Figure 131

ACTIVATED CARBON COLUMN PERFORMANCE
TOC REMOVAL FROM UNTREATED WASTEWATER AS A FUNCTION OF TOC APPLIED

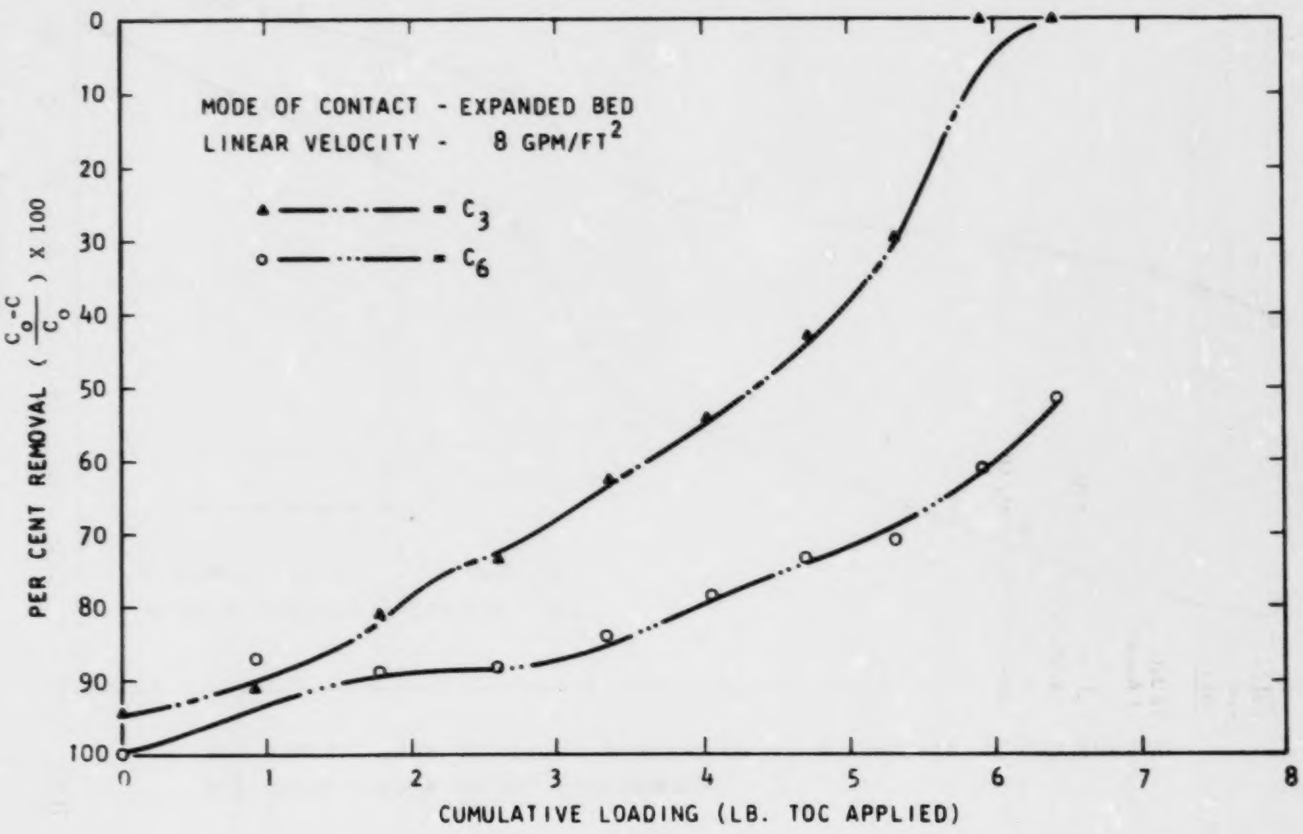


TABLE 45

SUMMARY OF TESTING

ITEM	Column Experiment No. 1	Column Experiment No. 2	Column Experiment No. 3	Column Experiment No. 4
(a) Period of Operation	28 July - 2 August	27 May - 15 June	3 July - 14 July	15 July - 23 July
(b) Mode of contact	Upflow expanded bed	Down Flow	Down Flow	Down Flow
(c) Influent	Raw wastewater	Primary Eff.	Secondary Eff.	Secondary Eff.
(d) No. Columns	six	six	three	three
(e) Flow Rate (gpm)	0.37	0.20	0.205	0.45
(f) Linear Velocity (gpm/ft^2)	8.068	4.36	4.50	9.80
(g) lbs carbon/column				
Column No. 1	5	5	5	5
2	5	5	5	5
3	5	5	5	5
4	5	5		
5	5	5		
6	5	5		
TOTAL	30	30	15	15
(h) Volume/Column (ft^3)				
Column No. 1	0.175	0.175	0.175	0.175
2	0.175	0.175	0.175	0.175
3	0.175	0.175	0.175	0.175
4	0.175	0.175		
5	0.175	0.175		
6	0.175	0.175		
TOTAL	1.050	1.050	0.525	0.525
(i) Contact Time(min)				
Column No. 1	3.55	6.43	6.27	2.88
2	3.55	6.43	6.27	2.88
3	3.55	6.43	6.27	2.88
4	3.55	6.43		
5	3.55	6.43		
6	3.55	6.43		
TOTAL	21.30	38.5	18.8	8.65

placed in the columns, with influent applied at a rate of 0.2 gpm providing a total contact time of approximately 38 minutes. The actual performance data in terms of the COD removal are presented graphically in Figure 132 and 133.

The performance of the carbon columns operating on primary effluent is in many respects similar to the results achieved with the untreated wastewater. Even though the removal curve more closely approximates the classical breakthrough diagram, leakage of adsorption resistant constituents still greatly exceeds release criteria. Almost immediately following initiation of the experiment, effluent COD consistently exceeded 150 mg/l with a BOD_5/COD ratio of approximately 0.5. Apparently, neutralization and primary clarification does not significantly effect the adsorption resistant compounds found in the untreated wastewater as column leakage per unit of contact time was determined to be essentially the same.

The plot of percent removal as a function of cumulative COD loading shown in Figure 133 serves to accentuate the results, for removals dropped rapidly below 90 percent following a loading of only 0.11 lb. COD/lb. carbon. However a classical breakthrough curve developed as the adsorption wave exited the final column. Carbon capacity at exhaustion reached approximately 0.5 lb COD/lb carbon.

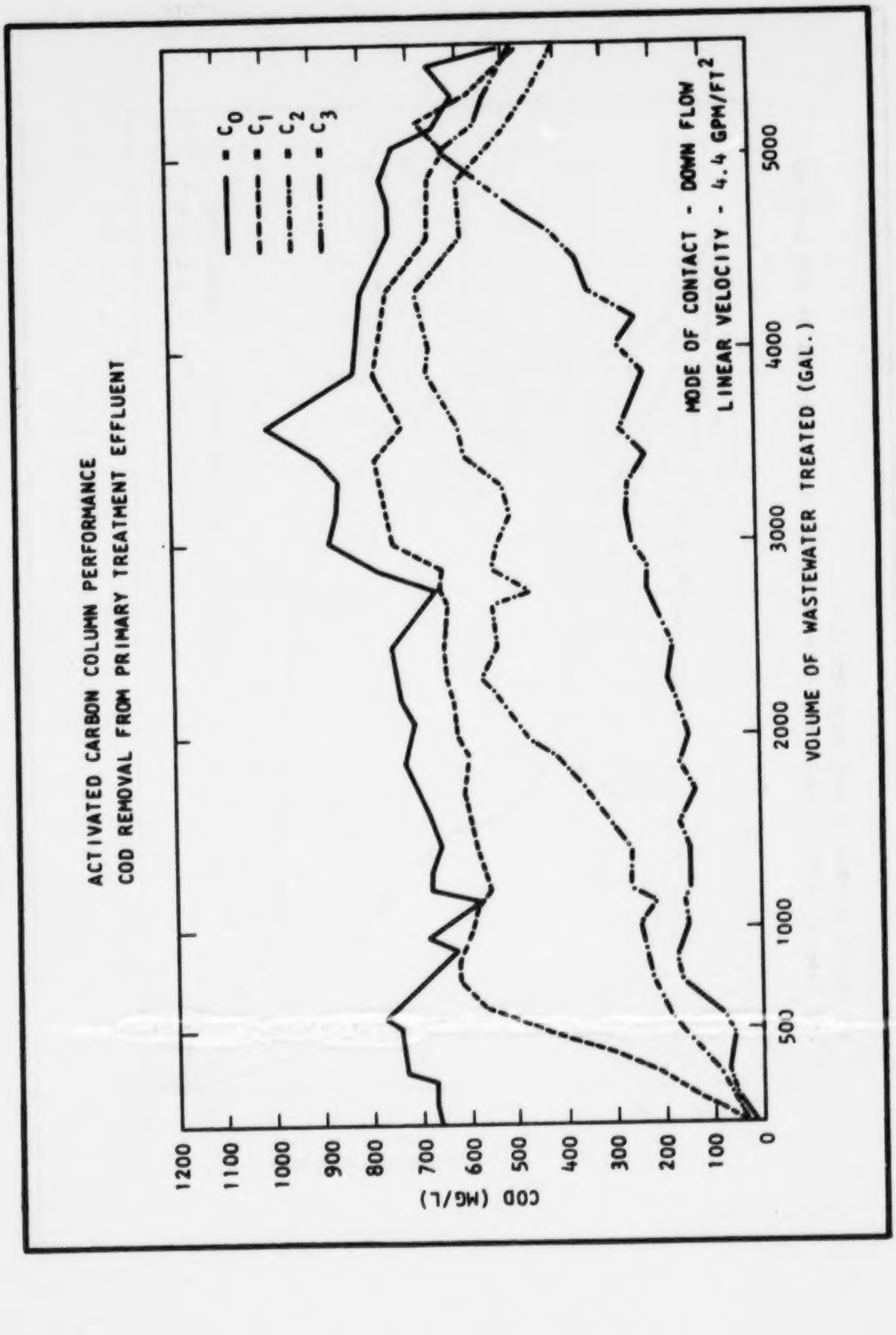
Results of Carbon Adsorption of Biologically Treated Effluent

Two separate downflow packed column experiments were performed upon the effluent from the biological Pilot Plant. Both experiments were conducted using three packed columns in series and a total of 15 pounds of Westvaco 12 x 40 mesh activated carbon. Linear flow velocities of 4.5 gpm/ft^2 and 9.8 gpm/ft^2 were maintained during the first and second experiments respectively. Empty bed volume contact times were respectively 18.8 and 8.7 minutes.

As shown in Figure 134, BOD_5 concentrations in the final column effluent during the first experiment never exceeded 10 mg/l. This can, in part, be attributed to the relatively dilute nature of the effluent. However, a considerable reduction in BOD_5 was accomplished. Evidently, the biological pre-treatment did not render the wastewater more amenable to carbon adsorption, but merely removed a large enough portion of the adsorption resistant compounds to reduce column leakage to an acceptable level.

In order to achieve a breakthrough at the projected release criteria of 20 mg/l BOD_5, throughput was increased by approximately a factor of two for the second experimental run. This objective was accomplished although complete exhaustion

Figure 132



ACTIVATED CARBON COLUMN PERFORMANCE
COD REMOVAL FROM PRIMARY TREATMENT EFFLUENT

MODE OF CONTACT - DOWN FLOW
LINEAR VELOCITY - 4.4 GPM/FT^2

Figure 133

ACTIVATED CARBON COLUMN PERFORMANCE
COD REMOVAL FROM PRIMARY TREATMENT EFFLUENT AS A FUNCTION OF COD APPLIED

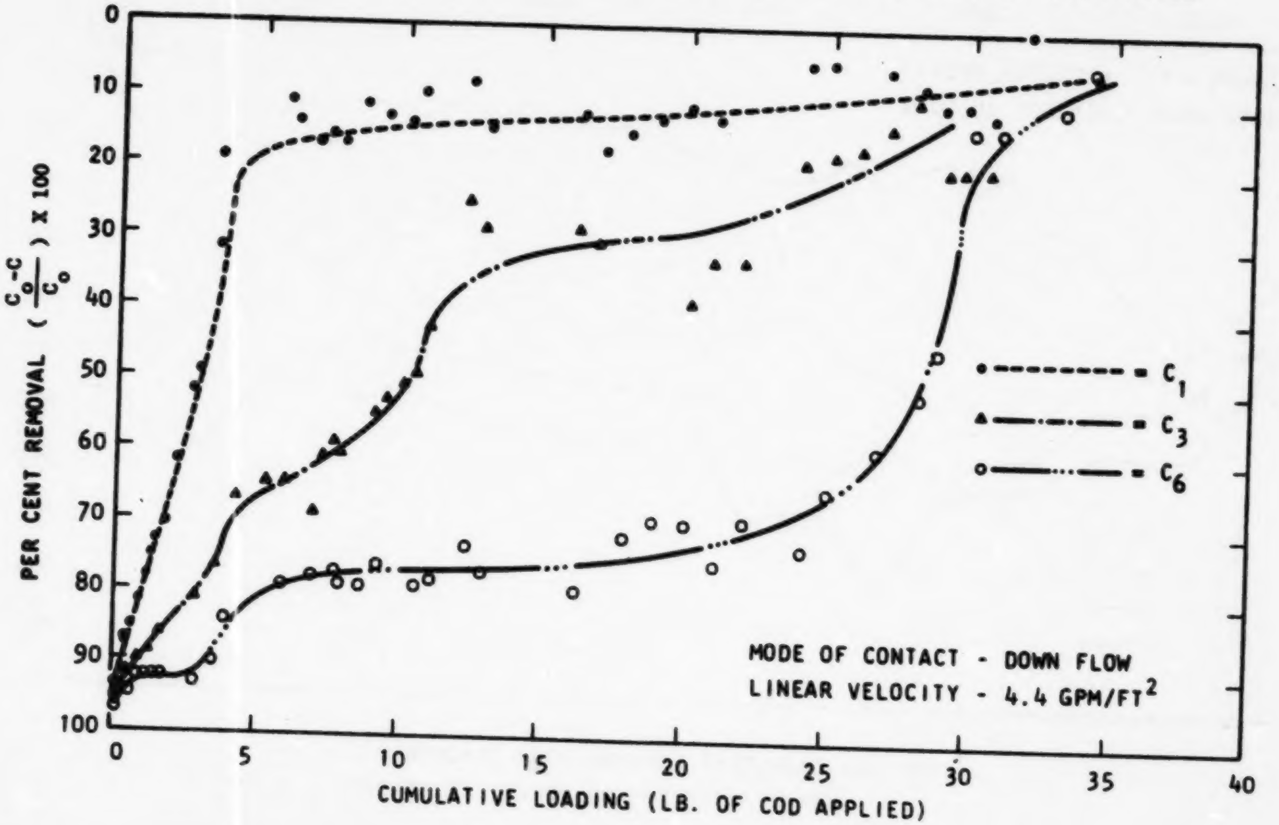
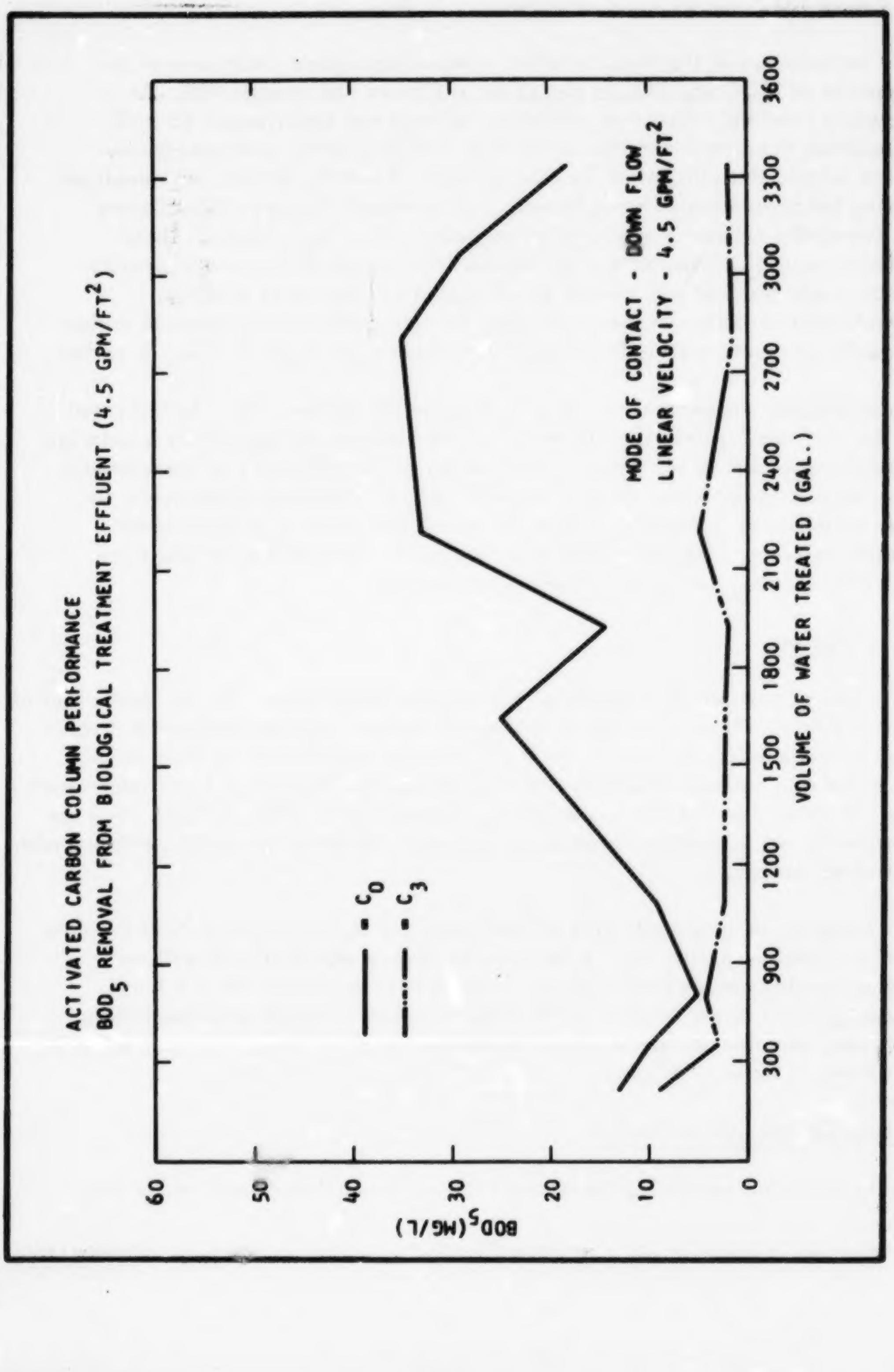


Figure 134



ACTIVATED CARBON COLUMN PERFORMANCE

BOD_5 REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (4.5 GPM/FT^2)

was never attained as BOD_5 removal remained above 60 percent, as shown in Figure 135.

The performance of the three columns in removing organic contaminants as measured by COD and TOC is presented in Figures 136 through 139. As shown, a relatively low level of column leakage was experienced for both parameters throughout the first experiment with adsorption zone emergence noted only in the effluent of the first column. A similar pattern was developed during the second experiment; however, contaminant loading was sufficient to essentially exhaust the adsorptive capacity of the lead column. These effects are further documented by Figures 140 through 143 in which percent contaminant removal was plotted as a function of cumulative loading. Graphical integration of the areas under the adsorption curves revealed carbon capacity of approximately 0.7 lbs COD/lb carbon and 0.25 lbs TOC/lb carbon.

Color removal performance is shown in Figures 144 through 146. As indicated in the first two Figures, the effluent from the columns was essentially a colorless fluid (irrespective of the extent of coloration of the influent) and breakthrough with respect to color was never achieved. Actual treatment performance for both experiments is shown in Figure 146 where the color of the biologically treated effluent, a distinct greenish yellow with a dominant wave length of 575 millimicrons, was almost completely removed.

Summary:

In summary, the pertinent results of this series of experiments was the determination of a highly significant leakage of adsorption resistant compounds when activated carbon was applied for the treatment of untreated wastewaters or those having received only primary treatment and neutralization. Perhaps of equal importance, was the discovery that the wastewater constituents responsible for coloration are apparently not adsorption resistant and are easily removed on contact with granular activated carbon.

The experiments conducted with effluent from the biological Pilot Plant indicate that a workable facility can be designed to remove essentially all effluent coloration and reduce other organic contaminant concentrations to a level acceptable for direct release to the Delaware River. Results and conclusions obtained from this test series were verified by the pilot-scale testing of effluent polishing by carbon adsorption.

Pilot Scale Carbon Column Studies

Pilot scale activated carbon studies were performed to supplement and verify the

Figure 135

ACTIVATED CARBON COLUMN PERFORMANCE
BOD$_5$ REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (9.8 GPM/FT^2)

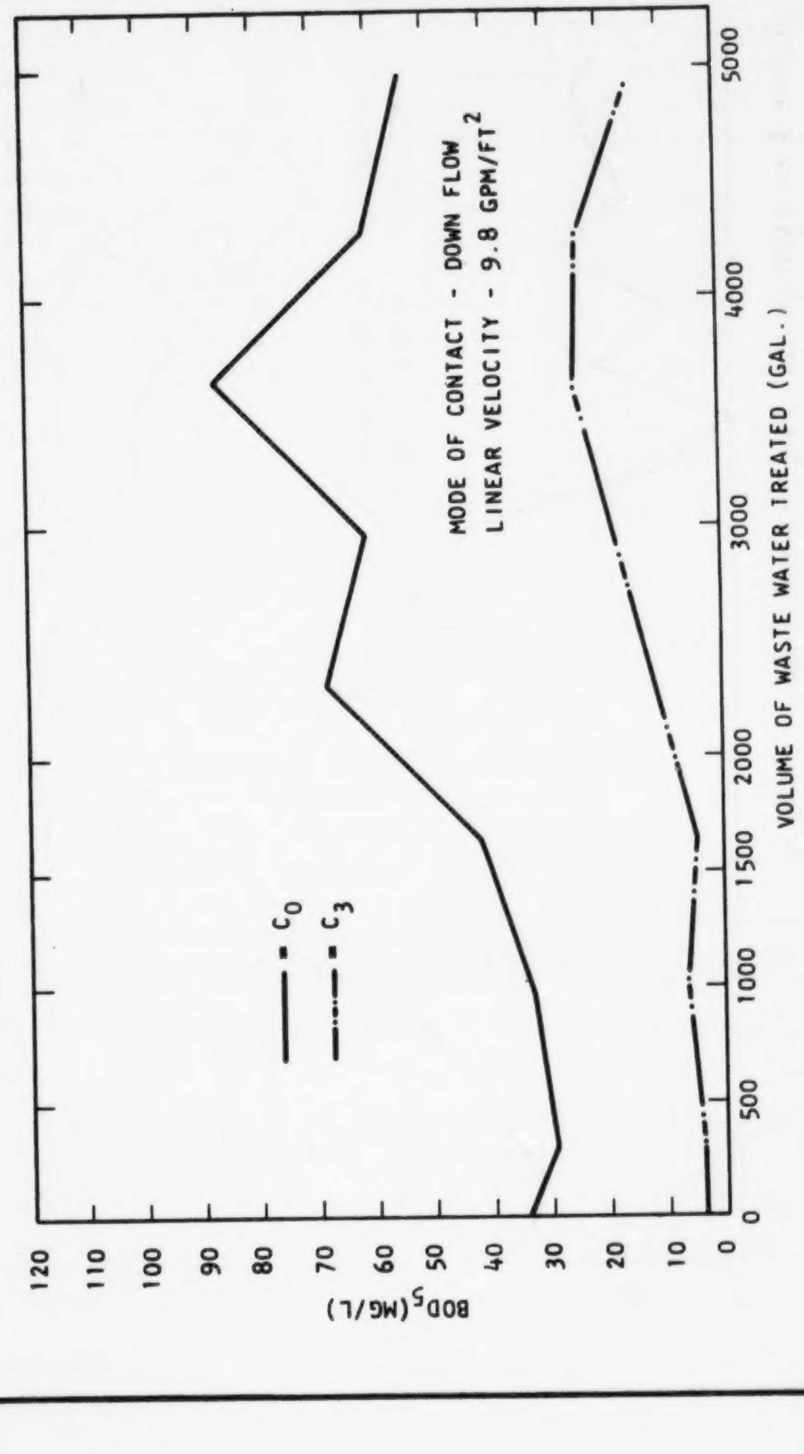
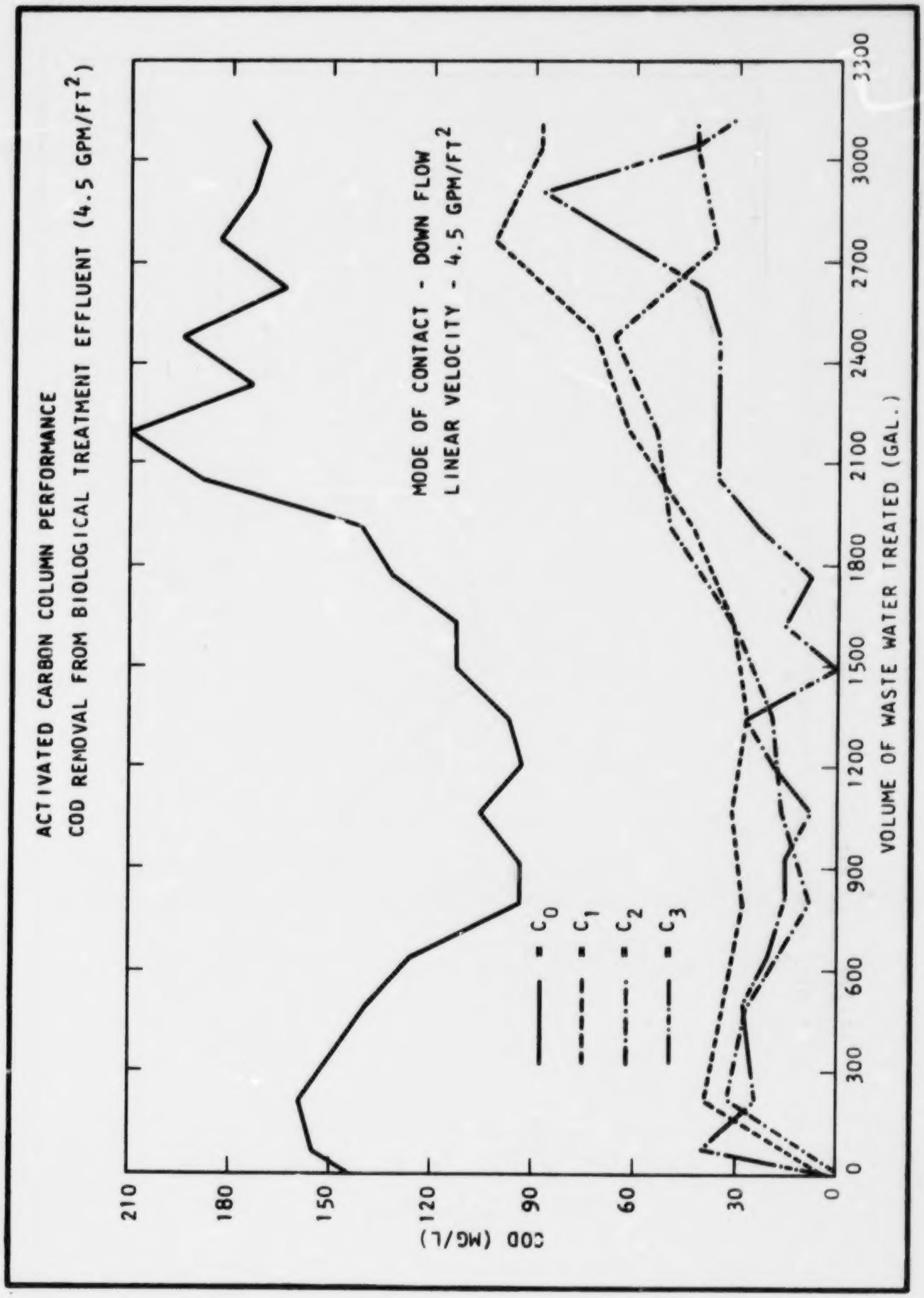


Figure 136



ACTIVATED CARBON COLUMN PERFORMANCE
COD REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (4.5 GPM/FT^2)

Figure 137

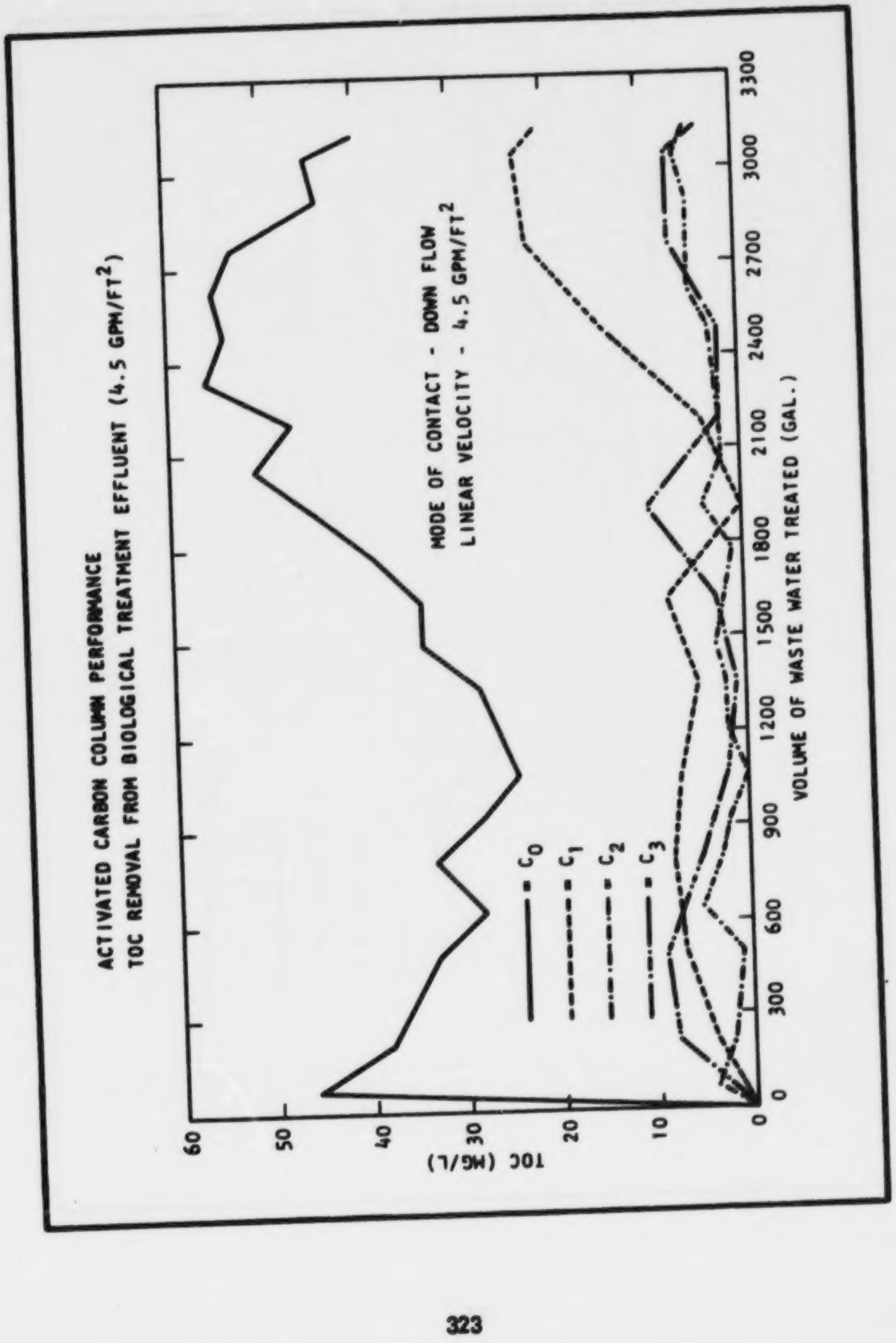
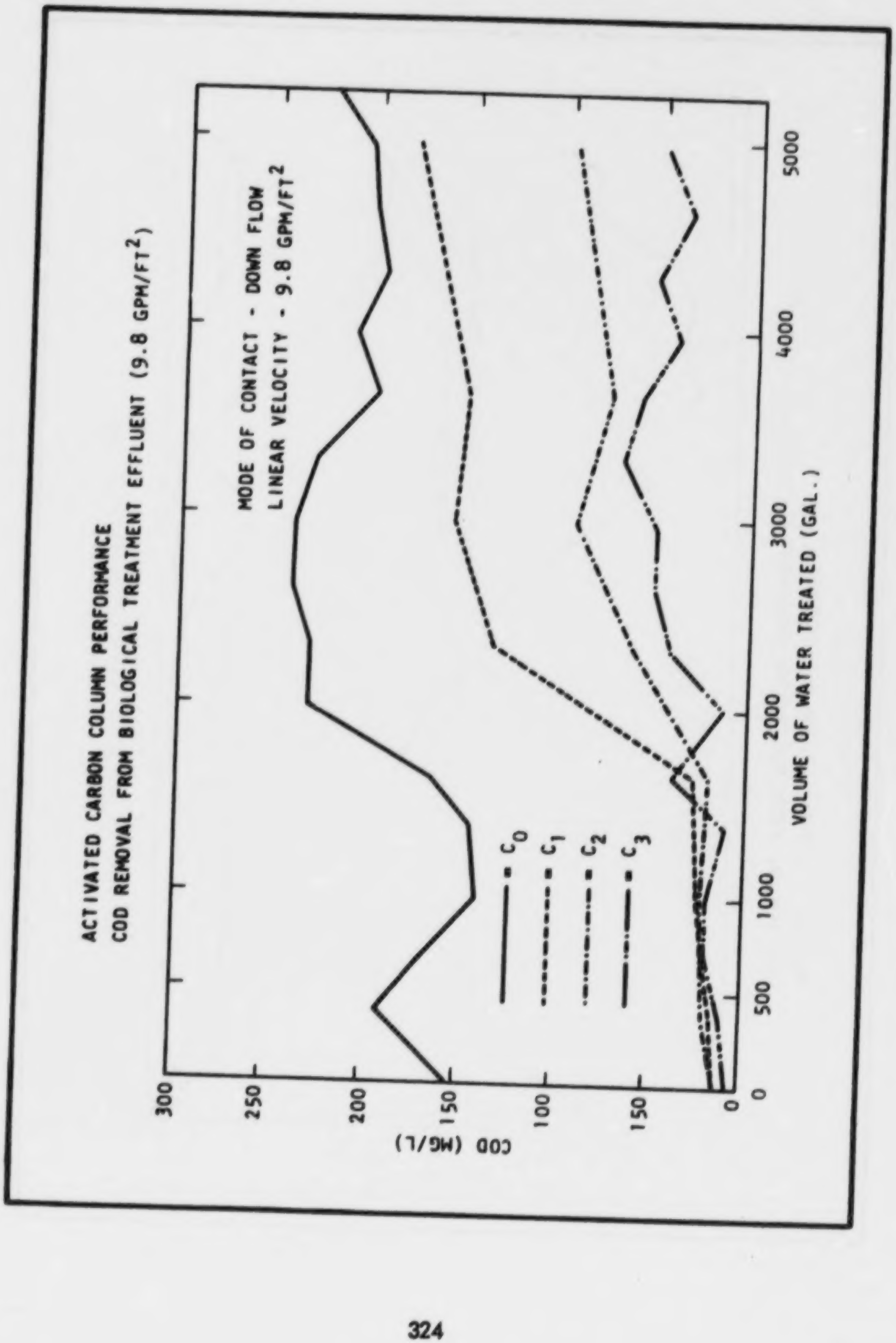
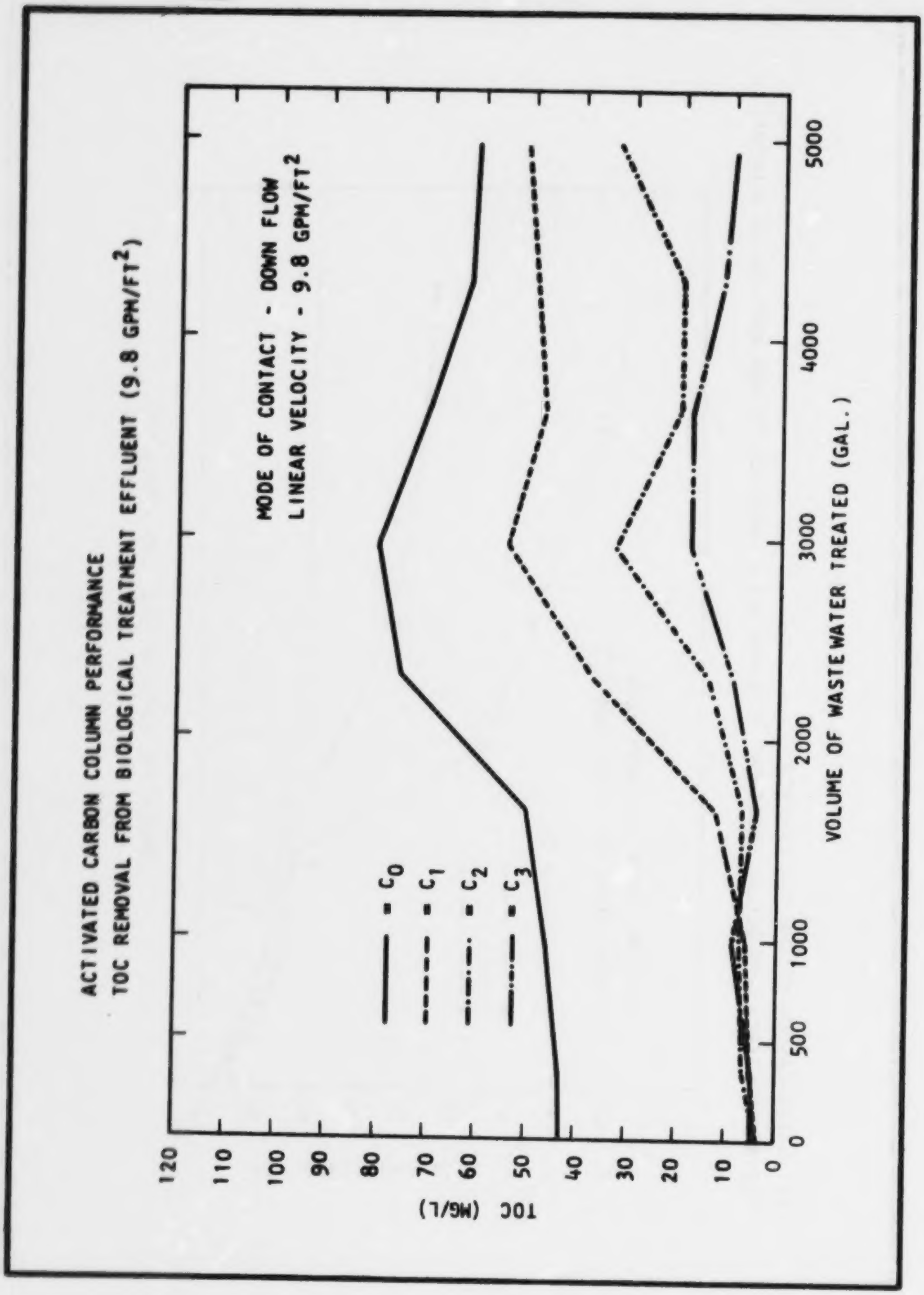

ACTIVATED CARBON COLUMN PERFORMANCE
TOC REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (4.5 GPM/FT^2)

Figure 138



ACTIVATED CARBON COLUMN PERFORMANCE
COD REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (9.8 GPM/FT^2)

Figure 139



ACTIVATED CARBON COLUMN PERFORMANCE
TOC REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT (9.8 GPM/FT^2)

Figure 140

ACTIVATED CARBON COLUMN PERFORMANCE

COD REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT AS A FUNCTION OF COD APPLIED (9.8 GPM/FT^2)

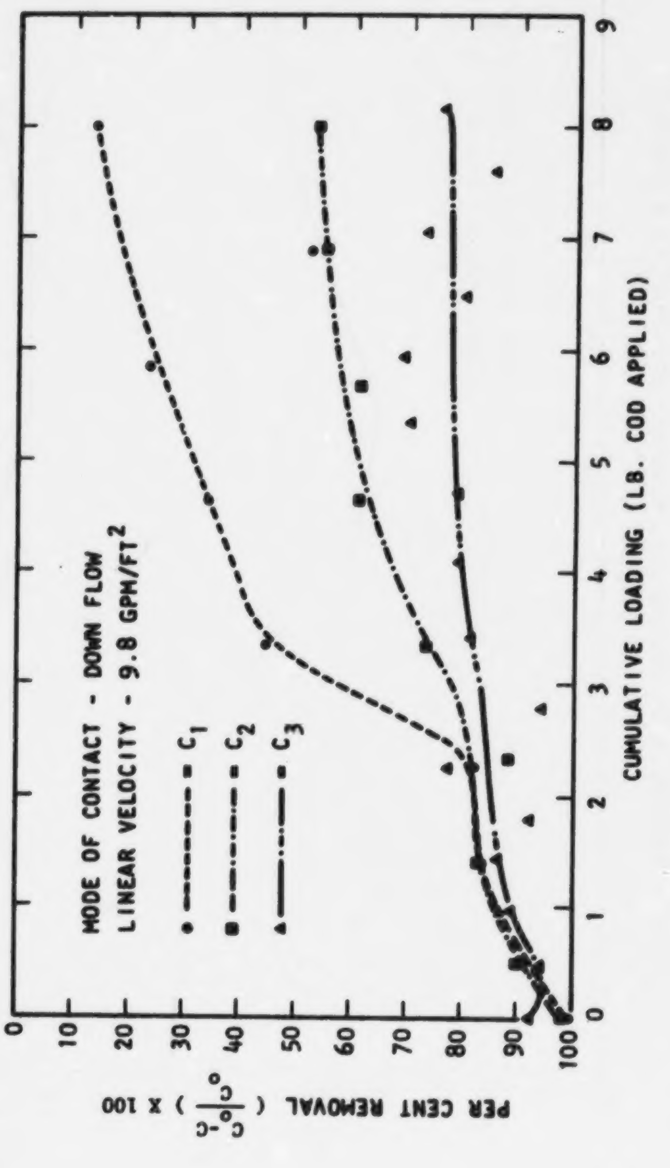
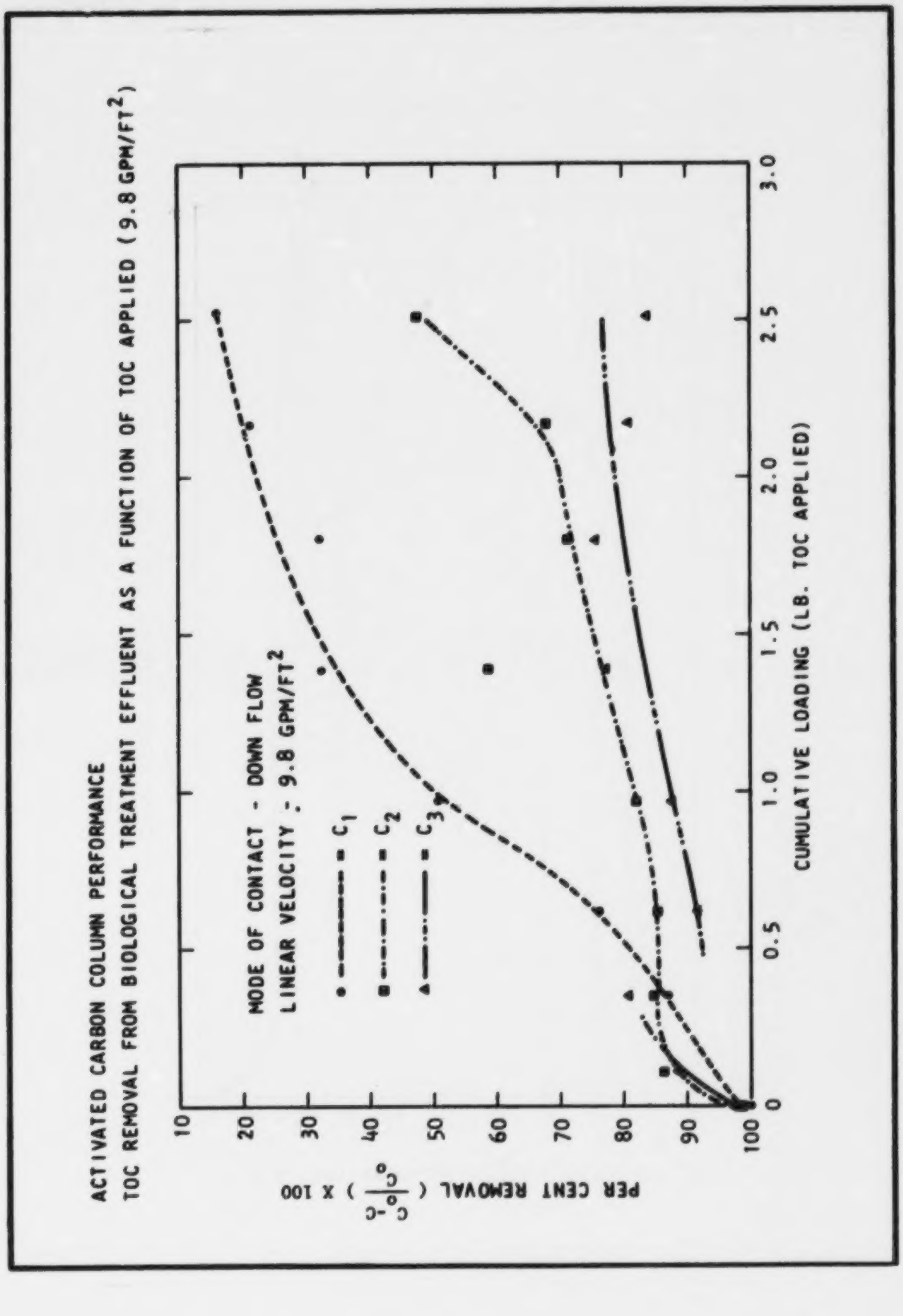


Figure 141



ACTIVATED CARBON COLUMN PERFORMANCE
TOC REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT AS A FUNCTION OF TOC APPLIED (9.8 GPM/FT^2)

Figure 142

ACTIVATED CARBON COLUMN PERFORMANCE

COD REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT AS A FUNCTION OF COD APPLIED (4.5 GPM/FT^2)

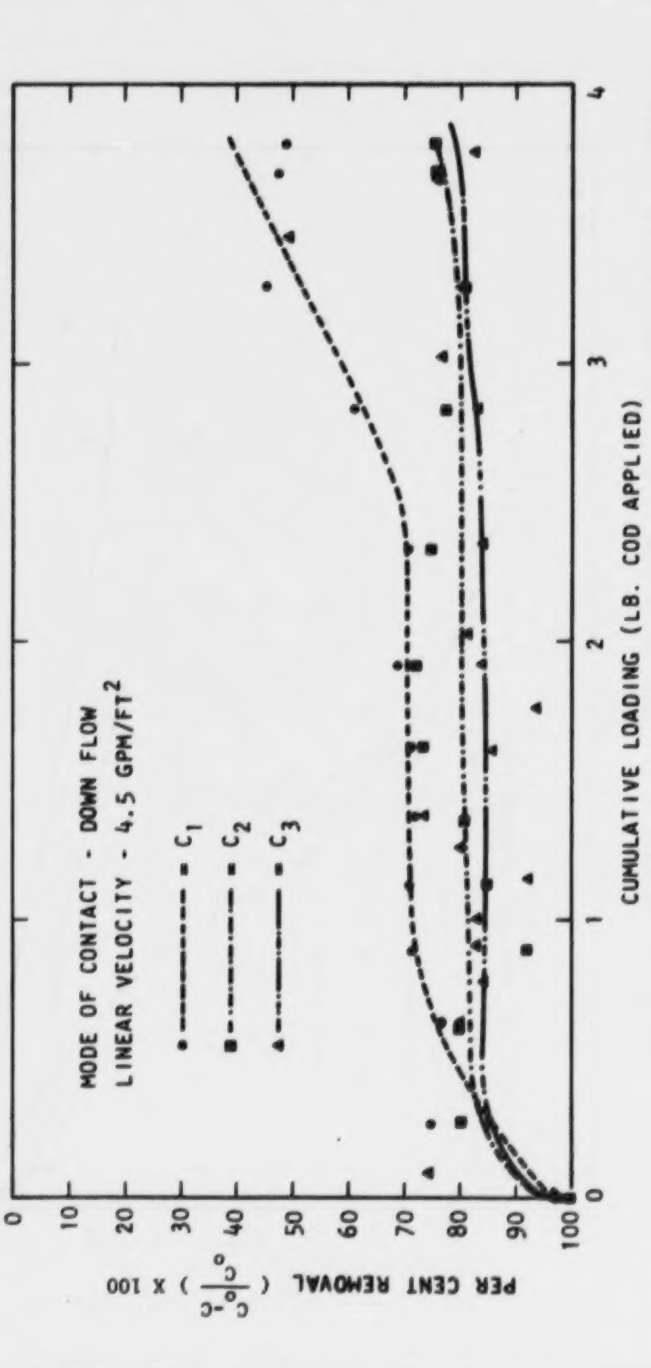
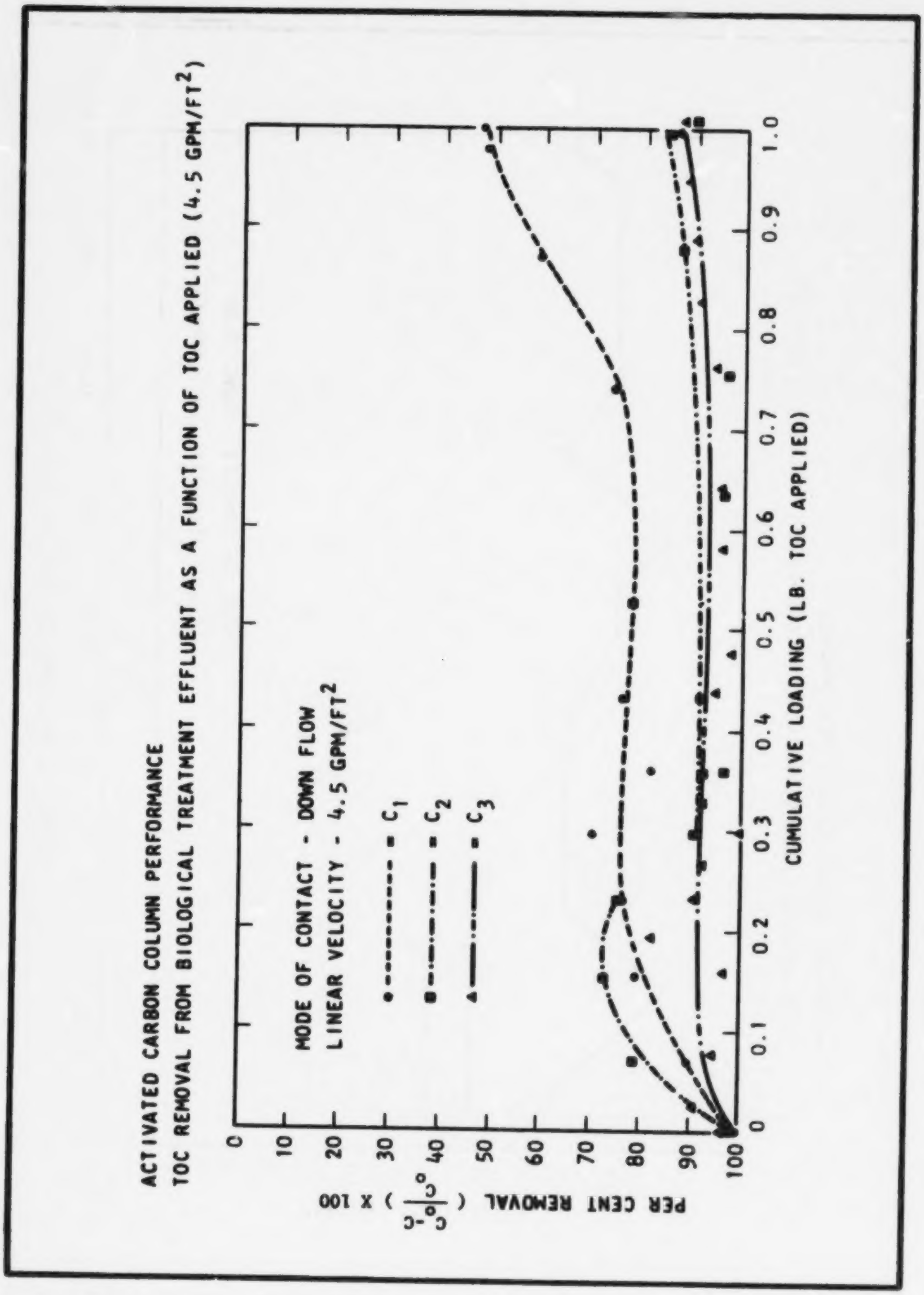


Figure 143



ACTIVATED CARBON COLUMN PERFORMANCE
TOC REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT AS A FUNCTION OF TOC APPLIED (4.5 GPM/FT^2)

Figure 144

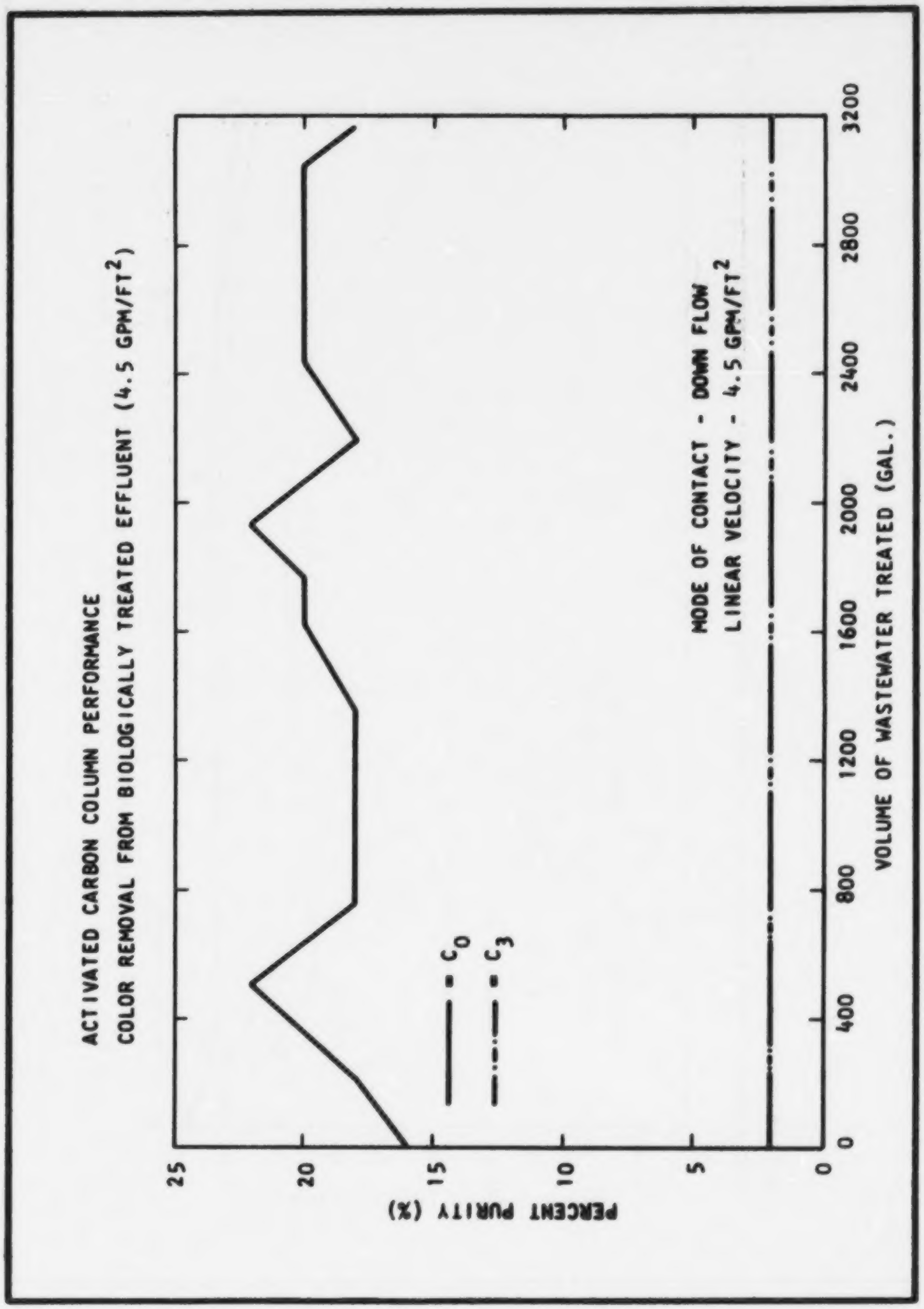
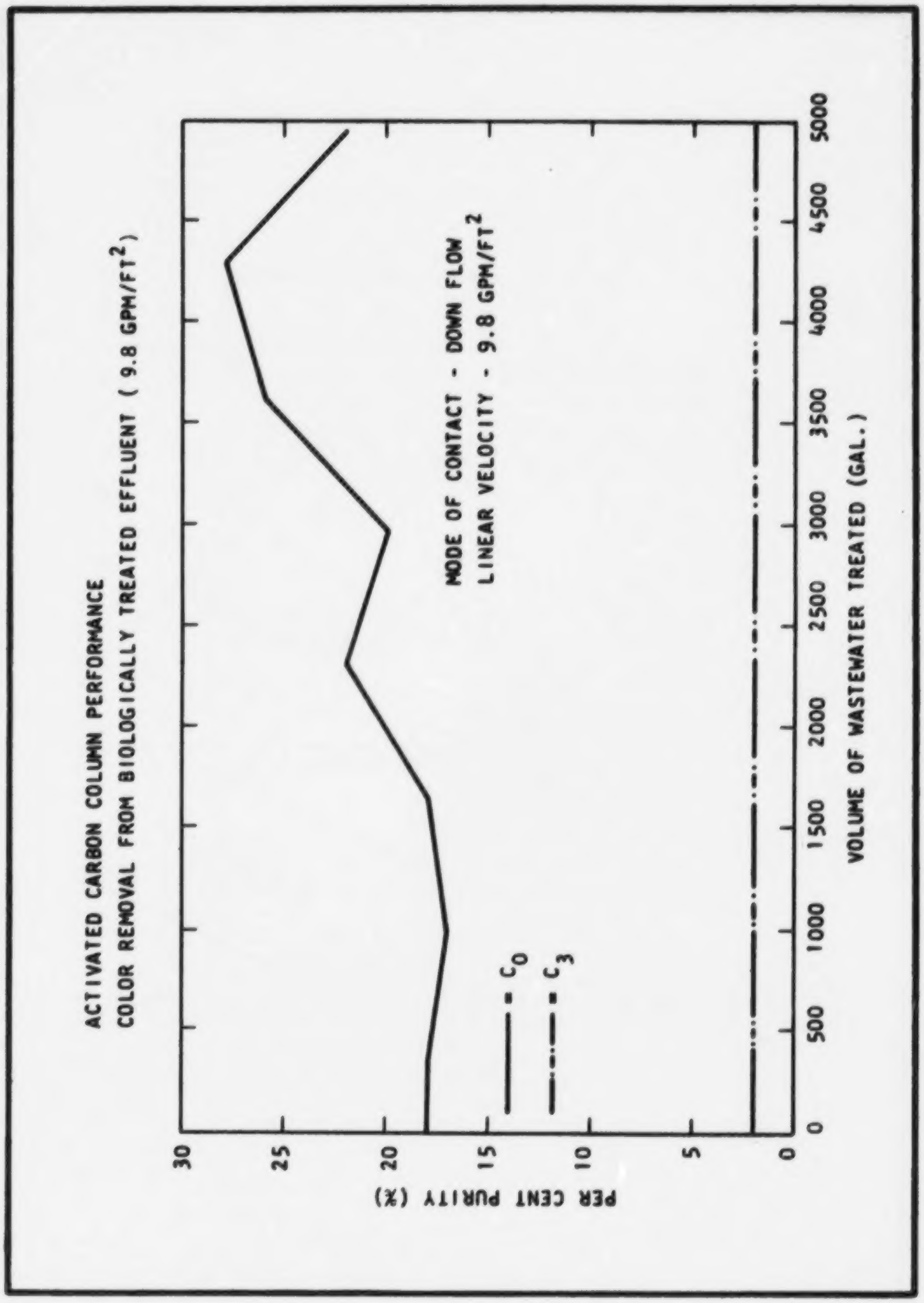
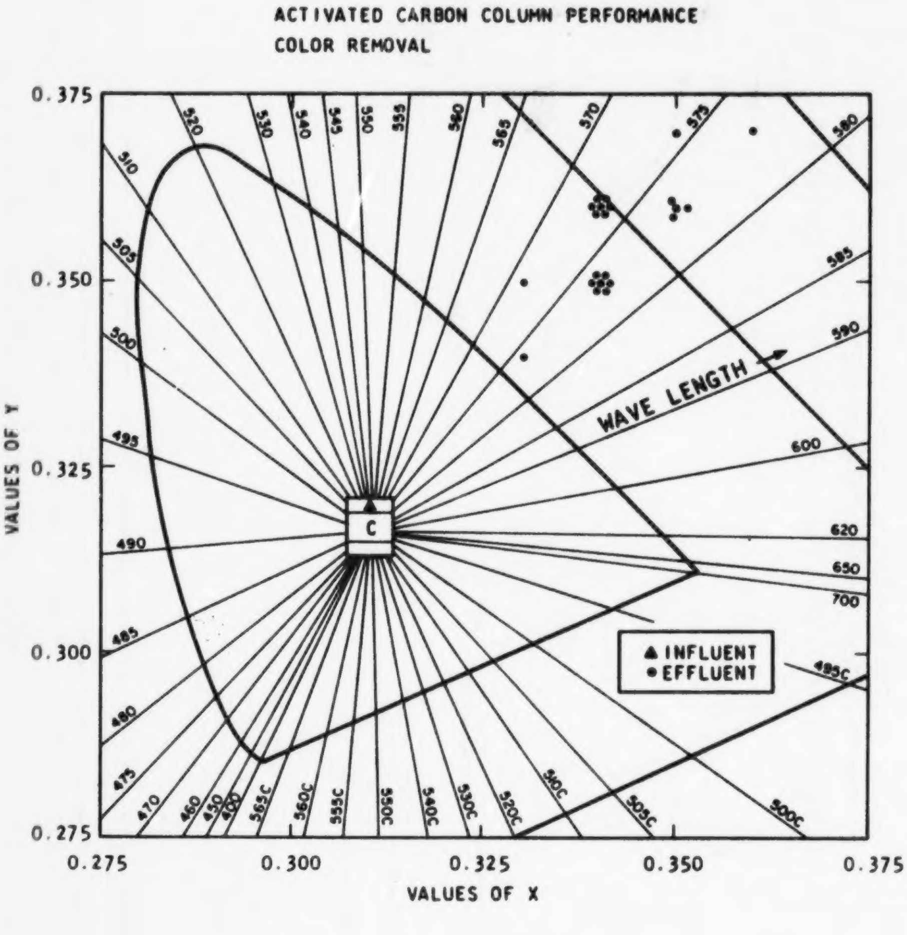

ACTIVATED CARBON COLUMN PERFORMANCE
COLOR REMOVAL FROM BIOLOGICALLY TREATED EFFLUENT (4.5 GPM/FT^2)
PERCENT PURITY (%)
25
20
15
10
5
0
VOLUME OF WASTEWATER TREATED (GAL.)
0
400
800
1200
1600
2000
2400
2800
3200
= C_0
= C_3
MODE OF CONTACT - DOWN FLOW
LINEAR VELOCITY - 4.5 GPM/FT^2

Figure 145



ACTIVATED CARBON COLUMN PERFORMANCE
COLOR REMOVAL FROM BIOLOGICALLY TREATED EFFLUENT (9.8 GPM/FT^2)

Figure 146


ACTIVATED CARBON COLUMN PERFORMANCE
COLOR REMOVAL
VALUES OF Y
VALUES OF X
0.375
0.350
0.325
0.300
0.275
0.275
0.300
0.325
0.350
0.375
WAVE LENGTH
C
▲ INFLUENT
● EFFLUENT
510
505
500
495
490
485
480
475
470
460
450
400
520
530
540
545
550
555
560
565
570
575
580
585
590
600
620
650
700
565C
560C
555C
550C
540C
530C
520C
510C
505C
500C
495C

data obtained from the bench scale columns. These studies were undertaken using a three foot in diameter upflow filter shell packed to a seven foot carbon bed depth. The column was piped to serve as an effluent polishing unit receiving the effluent from the pilot plant biological system. The carbon used in the column for these studies was Westvaco 12 x 40 mesh Nuchar, which was the same carbon used in the bench scale columns. Three different runs were performed in this test series by varying the hydraulic application rate. The performance of the column in terms of quality response was recorded, and the results are reported in this section. A diagram of the test column is shown in Figure 147.

Sampling and Analysis Schedule:

Sampling points established for this test series included the raw waste to the biological system, the biological effluent, or carbon column influent, and the effluent from the columns. Grab samples were obtained daily, the volume of throughput recorded, and the following analyses were performed:

a. biochemical oxygen demand
b. chemical oxygen demand
c. color
d. pH
e. phenols
f. MBAS
g. total carbon

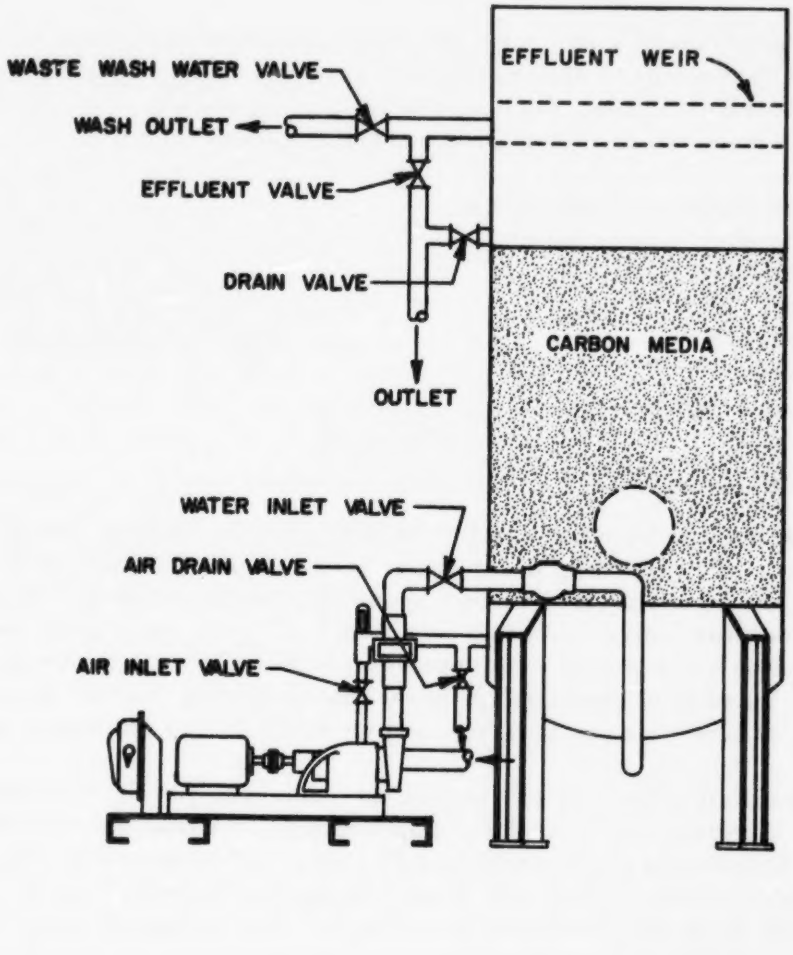
Discussion of Results:

Three separate runs were performed at various hydraulic loadings, but using the biologically treated effluent as the charge in each case. The quality of the effluent from the biological treatment system was representative of what might be expected from summer operating conditions. Although the organic concentration would be higher during winter operation as previously noted, the geometry of the BOD and COD breakthrough curves observed during this test series indicates that summer conditions can be safely used for establishing a year-round design basis.

The three different test runs will be discussed individually. The observed data from each run is presented in tabular and graphical form, the results compared to those from the bench scale studies, and the selection of design parameters finalized. These parameters are in turn used for establishing the conceptual design of an effluent polishing system using carbon columns. This conceptual design is presented in Section VII and serves as the basis for estimating capital and operating costs which are included in Section VIII.

Figure 147

PILOT SCALE CARBON COLUMN



WASTE WASH WATER VALVE
WASH OUTLET
EFFLUENT VALVE
DRAIN VALVE
OUTLET
EFFLUENT WEIR
CARBON MEDIA
WATER INLET VALVE
AIR DRAIN VALVE
AIR INLET VALVE

Test Series No. 1 -

Test Conditions:

Wastewater Charge: Biologically treated effluent
Carbon Column: 1,300 lbs of Westvaco 12 x 40 mesh "Nuchar"
Applied Flow: 21.8 gpm
Linear Flow Velocity: 3.08 gpm/ft^2
Contact Time: 17.8 minutes

The results of Test Series No. 1 are tabulated in Table 46. The column effluent in terms of filtered COD as a function of cumulative throughput volume is plotted in Figure 148. It is noted that the first noticeable breakthrough occurred following a throughput of 100,000 to 120,000 gallons. At the corresponding COD concentration level of 80 mg/l, the cumulative loading to the carbon is approximately 0.2 lbs COD applied/lb carbon as seen in Figure 149. It is noted that the data generated from the bench scale columns compares favorably with that from the pilot scale columns with respect to cumulative loading.

The color removal in the carbon column as a function of volume throughput is plotted in Figure 150. It is observed that any apparent color breakthrough occurs long after COD breakthrough, which only confirms the results recorded during the bench scale studies.

Test Series No. 2 -

Test Conditions:

Wastewater Charge: Biologically treated effluent
Carbon Column: 1,300 lbs of Westvaco 12 x 40 mesh "Nuchar"
Applied Flow: 17.0 gpm
Linear Flow Velocity: 2.4 gpm/ft^2
Contact Time: 23 minutes

The results of Test Series No. 2 are tabulated in Table 47. The column effluent in terms of filtered COD as a function of cumulative throughput volume is plotted in Figure 151. Analysis of this plot indicates multi-phase breakthrough. This phenomenon is accentuated when the data are plotted in the format of percent COD removal as a function of volumetric throughput as shown in Figure 152. This Figure indicates that an apparent initial breakthrough, or "COD leakage," occurs immediately after initiating operation of the column. A secondary breakthrough occurs at approximately 30,000 gallons throughput, and a final breakthrough occurs at approximately 560,000 gallons

TABLE 46

ACTIVATED CARBON COLUMN RESULTS - 3.08 gpm/ft^2 (Q = 21.8 gpm)

Sample No.	Date	pH	COD* mg/l	BOD* mg/l	Color USPHS	Phenol mg/l	MBAS mg/l
601	5/3/71	2.2	405	183	832	4.5	1.98
636		7.1	186	32	1099	1.7	1.26
695		7.6	29	18	70	0.10	0.12
601	5/4/71	2.3	393	162	832	5.2	1.35
636		7.1	148	35	1099	0.45	-
695		7.0	45	25	70	0.095	-
601	5/5/71	2.4	419	146	727	4.05	2.08
636		7.2	126	25	1158	0.40	1.19
695		7.2	41	23	136	0.05	0.05
601	5/6/71	2.4	363	144	1311	3.65	-
636		7.3	102	31	855	0.40	-
695		7.2	37	18	263	0.12	-
601	5/7/71	2.4	518	170	998	10.10	-
636		7.1	180	20	962	0.30	-
695		7.0	65	17	135	0.09	-
601	5/8/71	2.1	565	223	-	6.85	-
636		7.1	244	36	-	0.46	-
695		6.9	77	26	-	0.05	-
601	5/9/71	2.2	426	227	1187	6.85	2.91
636		7.2	153	38	1358	0.46	1.68
695		7.0	72	27	212	0.05	0.16
601	5/10/71	2.3	399	205	859	6.55	2.45
636		7.2	177	43	1290	0.44	1.53
695		6.9	80	31	102	0.06	0.14
601	5/11/71	2.4	510	199	1008	4.45	1.98
636		7.3	181	39	2383	0.38	1.52
695		7.1	96	27	230	0.08	0.15
601	5/12/71	2.4	486	189	994	-	-
636		7.3	155	29	1840	-	-
695		7.1	92	22	84	-	-
601	5/13/71	2.6	403	199	939	7.2	-
636		7.2	155	48	1027	0.34	-
695		7.4	72	18	93	-	-
601	5/14/71	2.4	500	198	-	-	1.89
636		7.1	204	43	-	-	0.99
695		7.1	76	36	-	-	-

* COD and BOD values based on filtered samples

601 = raw waste
636 = biological effluent (influent to carbon column)
695 = effluent from carbon column

TABLE 46 cont'd.

ACTIVATED CARBON COLUMN RESULTS - 3.08 gpm/ft^2 (Q = 21.8 gpm)

Sample No.	Date	pH	COD* mg/1	BOD* mg/1	Color USPHS	Phenol mg/1	MBAS mg/1
601	5/15/71	2.4	315	135	517	5.05	2.24
636		7.0	118	30	601	0.32	0.91
695		6.7	83	26	154	0.06	0.15
601	5/16/71	2.6	427	197	578	6.55	1.89
636		6.9	142	37	523	0.32	1.08
695		7.0	87	35	117	0.11	0.20
601	5/17/71	2.6	413	182	709	6.30	1.30
636		7.0	165	33	691	6.36	1.01
695		7.0	83	23	148	0.13	0.20
601	5/18/71	2.3	336	148	1064	4.05	-
636		7.1	154	28	1395	0.33	-
695		6.8	79	18	197	0.11	-
601	5/19/71	2.2	406	184	1547	6.75	-
636		6.9	141	31	1260	0.36	-
695		6.8	102	22	271	0.15	-
601	5/20/71	2.3	398	-	1238	-	-
636		7.4	168	-	1414	-	-
695		7.1	94	-	346	-	-
601	5/21/71	2.3	477	228	1322	6.10	-
636		7.3	131	26	1488	0.23	-
695		7.3	89	22	186	0.15	-
601	5/22/71	2.1	347	212	621	6.75	-
636		7.2	126	29	629	0.26	-
695		- - - - - - - - - NO SAMPLE - - - - - - - -					
601	5/23/71	2.4	370	236	555	6.75	-
636		7.3	105	30	810	0.36	-
695		7.1	97	26	349	0.25	-
601	5/24/71	2.5	444	213	1099	8.70	-
636		7.0	142	30	752	3.15	-
695		7.0	92	21	405	0.25	-
601	5/25/71	2.5	360	160	1130	-	-
636		7.1	131	23	1030	-	-
695		7.0	76	17	546	-	-
601	5/26/71	2.4	396	120	1065	-	-
636		6.8	125	14	999	-	-
695		6.9	83	5	334	-	-

* COD and BOD values based on filtered samples.

601 = raw waste
636 = biological effluent (influent to carbon column)
695 = effluent from carbon column

TABLE 47

ACTIVATED CARBON COLUMN RESULTS - 2.4 gpm/ft^2 (Q = 17 gpm)

Sample No.	Date 1971	pH	COD mg/1	BOD mg/1	Color USPHS	Phenol mg/1	MBAS mg/1	TC mg/1	Gal/ day	Total Gal
601 636 695	6/10	2.2 7.3	344 117 (18)	122 17.5			2.55 1.14	135 63		
601 636 695	6/11	2.2 7.3	327 121 (40)	121 13.9	1229 722	4.6 .12	3.17 1.41	90 68	16,400	[8,200] 16,400
601 636 695	6/12	2.1 7.3	365 104 (16)	138 13.8	1382 597	5.0 Trace	3.72 1.75	109 62	23,600	[28,200] 40,000
601 636 695	6/13	2.15 6.85	310 149 (28)	122 11.3	438 683	6.7 .05	4.20 1.01	129 56	18,000	[49,000] 58,000
601 636 695	6/14		352 128 (35)	99 15.6	714 662	4.8 0.7	4.4 2.24	117 87	24,000	[70,000] 82,000
601 636 695	6/15	2.4 7.15	356 96	119 14.7	1280 859	5.7 1.6	3.5 2.86	129 56	22,000	[93,000] 104,000
601 636 695	6/16		(53)						26,000	[117,000] 130,000
601 636 695	6/17	2.2 6.8	184 104 (54.5)	148 10.2	465 739	5. .03	4.08 2.80	126 48	25,900	[142,900] 155,900
601 636 695	6/18	2.25 7.1	424 112 (48)	138 4.8	1241 694	8.8 0.4		120 50	24,100	[167,900] 180,000
601 636 695	6/19	2.2 7.0							22,300	[191,000] 202,300
601 636 695	6/20	2.3 7.1		165 13.1					19,700	[212,000] 222,000
601 636 695	6/21	2.3 7.15	373 115	174 22	1114 686	7.5 0.3	3.8 3.4	120 41	21,000	[232,500] 243,000
601 636 695	6/22	2.3 6.55	414 125 (68)*	197 11.8	1592 1548	1.6 0.25	2.66 2.73	147 35	24,000	[255,000] 267,000
601 636 695	6/23	2.45 7.4	382 102 (77)*	150 7.3	1698 712	8.0 0.25	3.3 3.0		26,800	[280,400] 293,800
601 636 695	6/24	2.55 7.4	263 120 (56)	103 9.6	1276 911	7.8 0.25	4.2 3.5	69	26,200	[306,900] 320,000

601 = raw waste
636 = biological effluent (influent to carbon column)
695 = effluent from carbon column

( ) = average of 4-6 hr grab samples (filtered)
( )*= average of 4-6 hr grab samples (unfiltered)
[ ] = total volumetric throughput at midpoint of daily sampling period

TABLE 47 (cont)

ACTIVATED CARBON COLUMN RESULTS - 2.4 gpm/ft^2 (Q = 17 gpm)

Sample No.	Date 1971	pH	COD mg/l	BOD mg/l	Color UNITS	Phenol mg/l	MBAS mg/l	TC mg/l	Gal/ day	Total Gal
601	6/25	2.6	390	168	1233	6.0	5.3	141		
636		7.4	112	10.2	993	.15	3.5	44		[332,700]
695			(62)						25,400	
										345,400
601	6/26	3.4	347	105	1057	4.5	3.8	120		
636		7.15	93	8.5	753	.15	3.26	57		[357,700]
695			59					26	26,600	
			(64)							370,000
601	6/27	2.9	427		981	6.0	3.8	117		
636		7.15	110		1024	0.25	3.26	43		[383,350]
695		7.0	55		263	0.10	0.84	30	26,700	
										396,700
601	6/28									
636										[410,300]
695			(58)						27,200	
										423,900
601	6/29									
636										[436,750]
695			(70)						25,700	
										449,600
601	6/30									
636										[463,200]
695			(71)						27,200	
										476,800
601	7/1									
636										[488,100]
695									26,600	
										501,400
601	7/2	2.5	366	148	1128	4.2	2.86	111		
636		6.2	122	13.2	774	0.26	2.0	43		[513,700]
695		6.6	67	6.0	221	0.055	0.95	33	26,600	
										526,000
601	7/3	2.65	299	103	700	5.5	2.0	60		
636		6.65	115	13.8	676	0.21	2.06	38		[538,900]
695		6.7	65	6.1	254	0.05	0.85	32	25,800	
										551,800
601	7/4	3.0	333	133	1006	4.9	2.6	126		
636		7.2	107	13.2	746	0.17	2.0	50		[564,400]
695		7.1	50	6.0	301	0.055	1.11	35	25,200	
										577,000
601	7/5	2.5	347	165	933	4.75	2.86	132		
636		7.0	115	13.2	349	0.15	2.19	50		[589,300]
695		7.5	76	8.1	801	0.06	1.5	37	24,100	
										601,100
601	7/6	2.8	369	163	995	4.05	2.73	108		
636		7.3	111	7.5	606	0.13	2.41	48		[613,400]
695		7.2	64	6.0	225	0.065	1.11	32	24,600	
										625,700
601	7/7	3.0		86	1135	4.45	2.86	105		
636		7.0		8.7	219	0.115	2.12	48		[639,100]
695		7.2		6.3	240	0.09	1.33	31	26,900	
										652,600
601	7/8	2.85		151	769	4.75	2.51	99		
636		7.05		10.3	545	0.165	1.92	40		[665,500]
695		7.2		10.4	268	0.115	1.33	31	25,800	
										678,400
601	7/9	2.85	326	105	1106	4.1	4.28	75		
636		6.9	120	10.8	1001	0.7	3.50	34		[691,000]
695		7.0	94	12.8	399	0.365	2.80	29	25,300	
										703,700

601 = raw waste
636 = biological effluent (influent to carbon column)
695 = effluent from carbon column

( ) = average of 4-6 hr grab samples (filtered)
( )*= average of 4-6 hr grab samples (unfiltered)
[ ] = total volumetric throughput at midpoint of daily sampling period

TABLE 47 (cont)

ACTIVATED CARBON COLUMN RESULTS - 2.4 gpm/ft^2 (Q = 17 gpm)

Sample No.	Date 1971	pH	COD mg/1	BOD mg/1	Color USPHS	Phenol mg/1	MBAS mg/1	TC mg/1	Gal/ day	Total Gal
601	7/10									
636										
695		6.95		9.0	1007	0.195	2.07	41	23,800	[715,600]
										727,500
601	7/11	2.7	343	148		7.4	3.78			
636		7.05	136	10.8		0.26	2.60			
695		7.05	106	10.8		0.135	2.41		22,000	[738,500]
										749,500
601	7/12									
636										
695									25,300	[762,150]
										774,800
601	7/13	2.8	322	113	926	6.2	4.08	118		
636		7.3	127	4.2	929	0.19	2.94	43		
695		7.4	93	6.7	956	0.14	2.35	35	23,900	[786,750]
										798,700
601	7/14	2.7	305	136	1176	7.5		108		
636		7.15		5.4	682	.20	3.7			
695		7.15	47	7.7	350	.15	2.24		28,100	[812,770]
										826,800

601 = raw waste
636 = biological effluent (influent to carbon column)
695 = effluent from carbon column

( ) = average of 4-6 hr grab samples (filtered)
( )*= average of 4-6 hr grab samples (unfiltered)
[ ] = total volumetric throughput at midpoint of daily sampling period

Figure 148

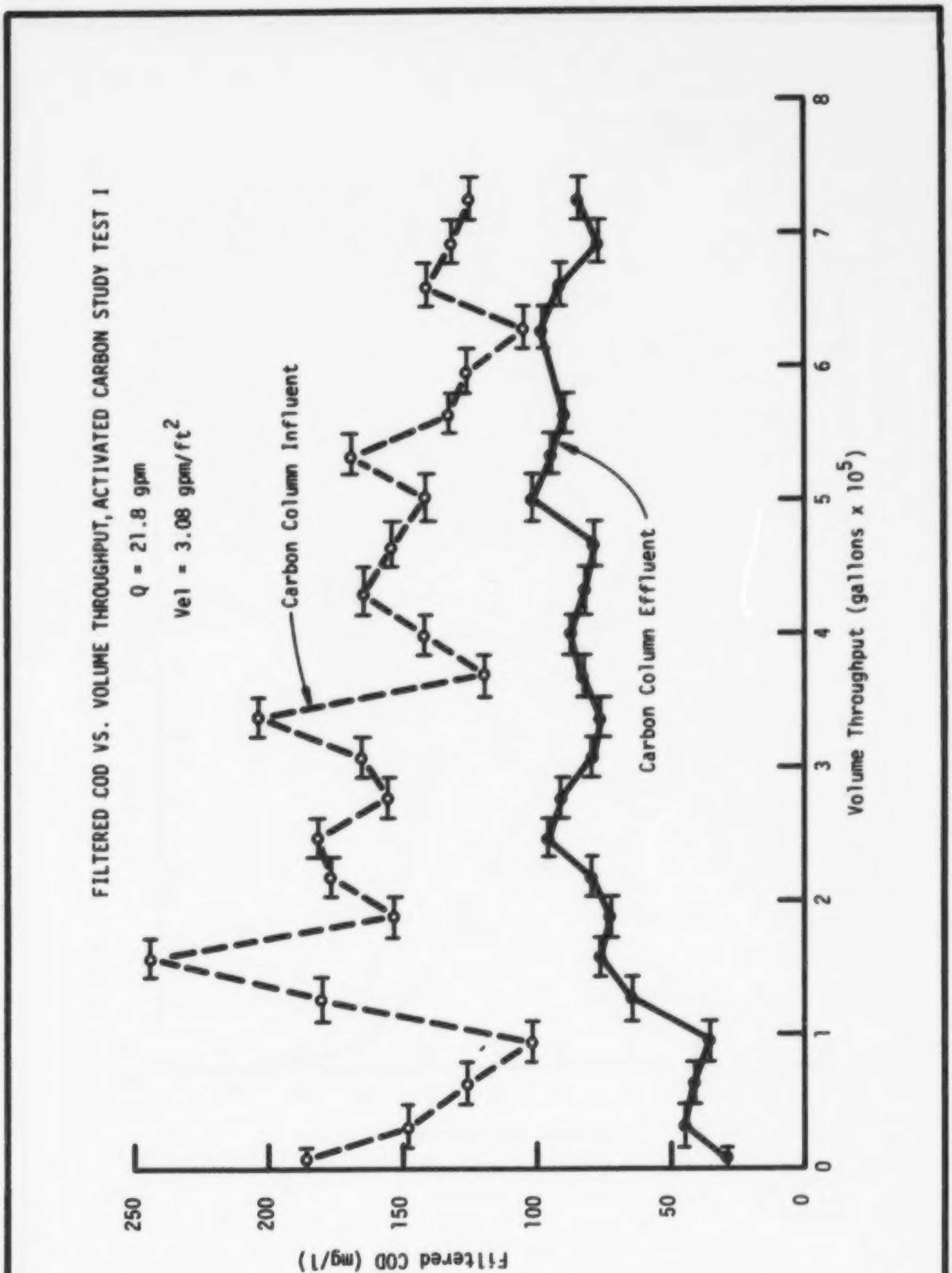

FILTERED COD VS. VOLUME THROUGHPUT, ACTIVATED CARBON STUDY TEST I
Q = 21.8 gpm
Vel = 3.08 gpm/ft^2
Carbon Column Influent
Carbon Column Effluent
Filtered COD (mg/l)
250
200
150
100
50
0
0
1
2
3
4
5
6
7
8
Volume Throughput (gallons x 10^5)

Figure 149

COD REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT AS A FUNCTION OF COD APPLIED
ACTIVATED CARBON STUDY TEST I (Based on Filtered COD Analysis)

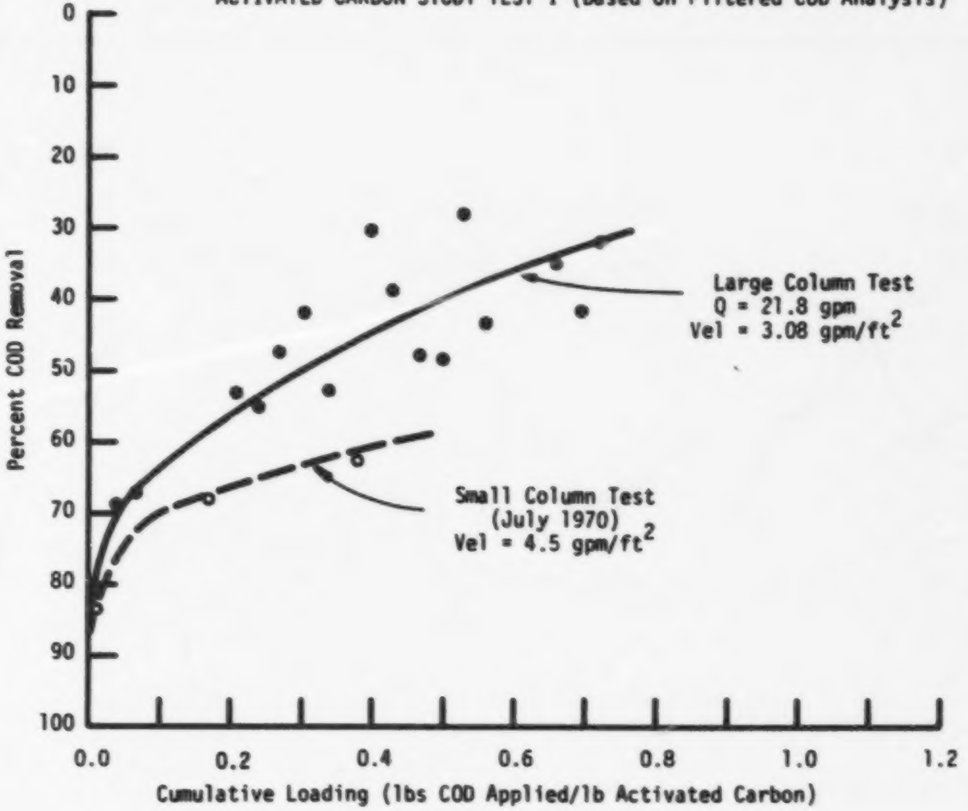


Figure 150

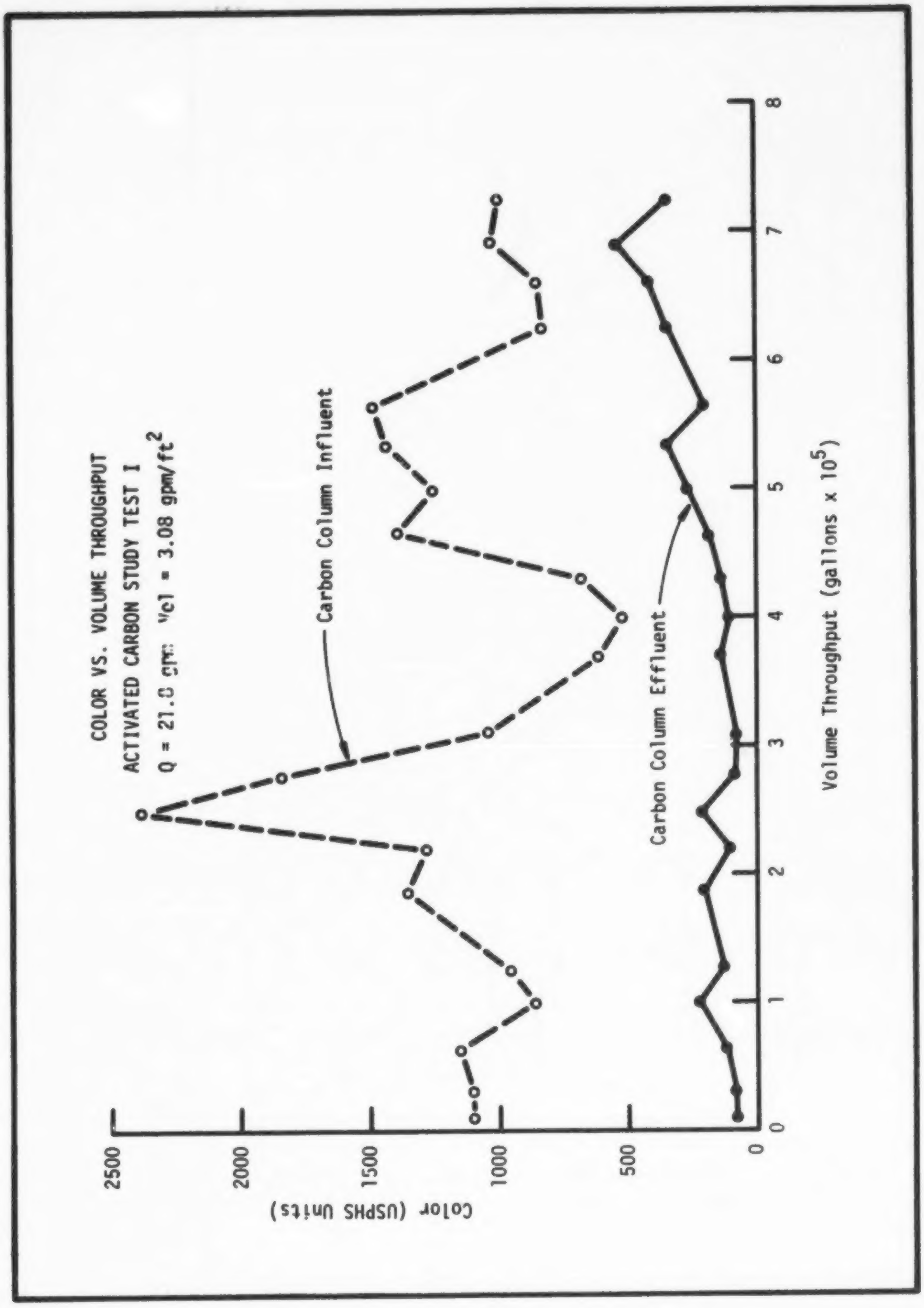

COLOR VS. VOLUME THROUGHPUT
ACTIVATED CARBON STUDY TEST I
Q = 21.8 gpm Vol = 3.08 gpm/ft^2
Carbon Column Influent
Carbon Column Effluent
Color (USPHS Units)
2500
2000
1500
1000
500
0
0
1
2
3
4
5
6
7
8
Volume Throughput (gallons x 10^5)

Figure 151

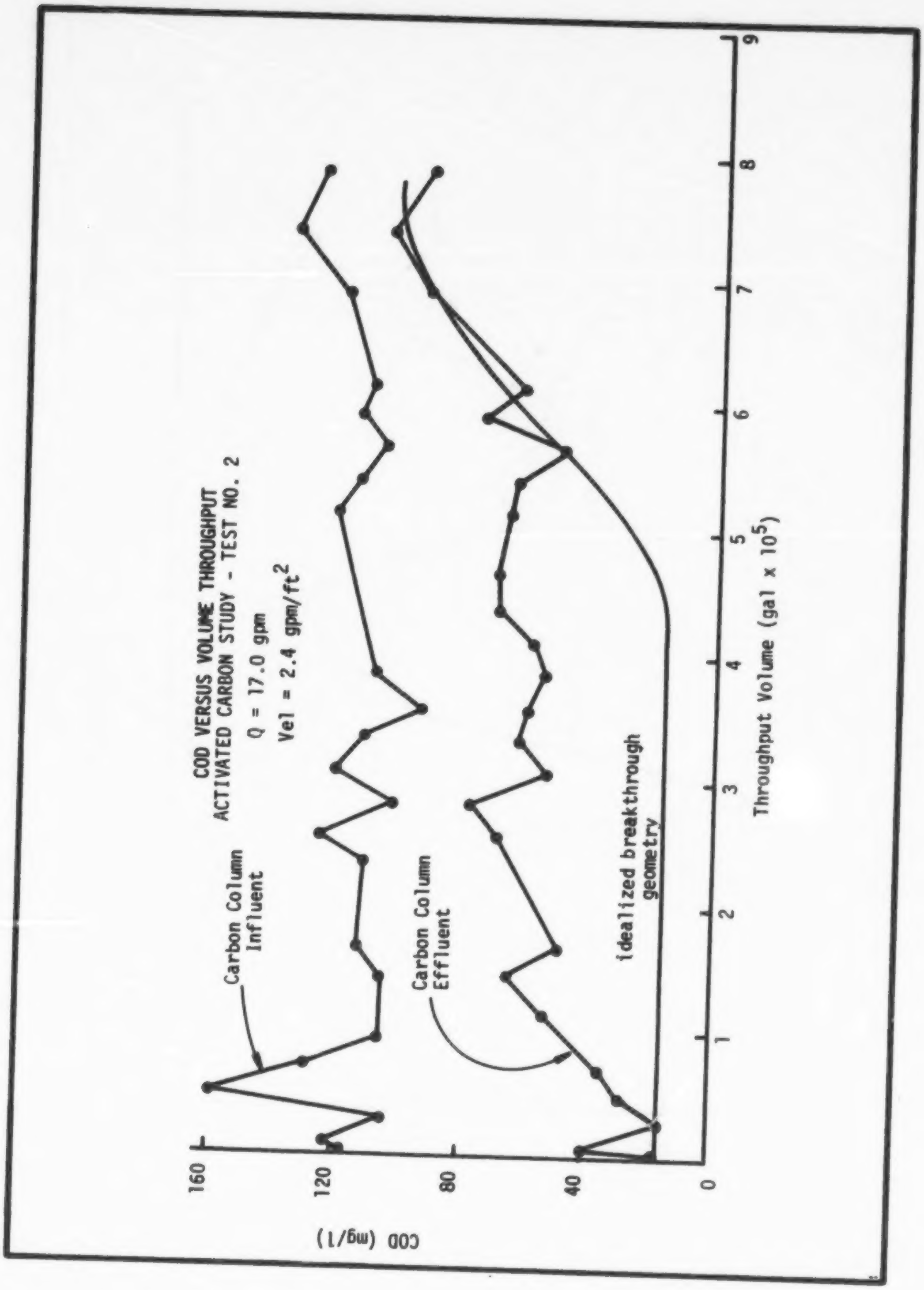
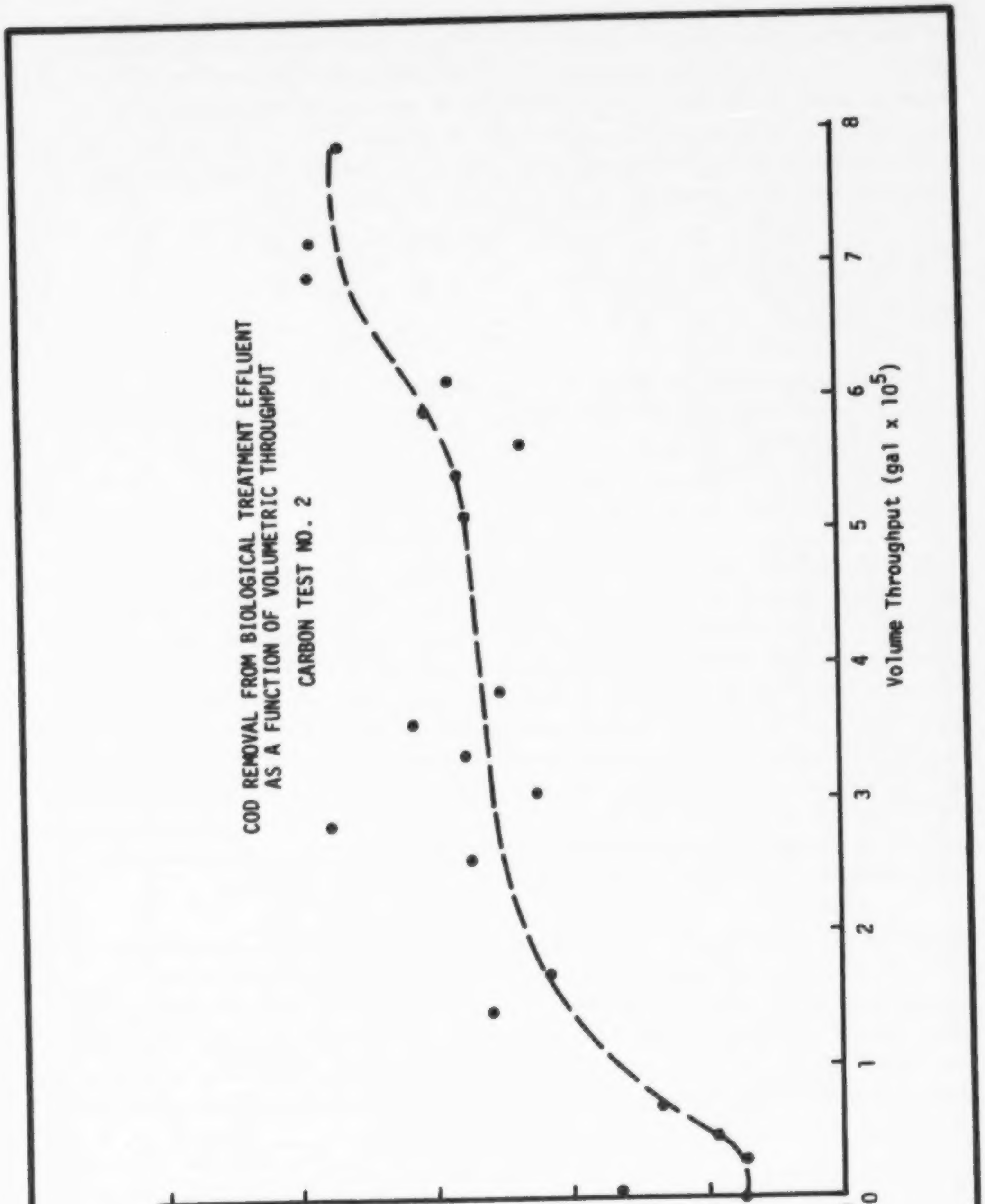

COD VERSUS VOLUME THROUGHPUT
ACTIVATED CARBON STUDY - TEST NO. 2
Q = 17.0 gpm
Vel = 2.4 gpm/ft^2
Carbon Column Influent
Carbon Column Effluent
idealized breakthrough geometry
COD (mg/l)
160
120
80
40
0
Throughput Volume (gal x 10^5)
1
2
3
4
5
6
7
8
9

Figure 152


COD REMOVAL FROM BIOLOGICAL TREATMENT EFFLUENT AS A FUNCTION OF VOLUMETRIC THROUGHPUT

CARBON TEST NO. 2

throughput. Based on this multiple breakthrough curve, a plot representing carbon capacity (lbs COD removed per lb of carbon applied) as a function of volumetric throughput is presented in Figure 153. Based on this graphical presentation, the carbon capacity at a COD breakthrough level of 80 mg/l is approximately 0.23 lbs COD removed per lb carbon and the capacity at exhaustion is 0.25 lbs COD removed per lb carbon. This is lower than the value reported during the bench scale studies, although the applied linear velocity was correspondingly lower.

Test Series No. 3 -

Test Conditions:

Wastewater Charge: Biologically treated effluent
Column Carbon: 1,040 lbs of Westvaco 12 x 40 mesh "Nuchar"
Applied Flow: 28 gpm
Linear Flow Velocity: 4 gpm/ft^2
Contact Time: 14 minutes

The results of Test Series No. 3 are tabulated in Table 48. The column effluent in terms of filtered COD as a function of cumulative throughput volume is plotted in Figure 154. It is observed from this figure that a significant COD leakage occurred immediately after beginning the run, then the concentration remained at or below 100 mg/l until almost 400,000 gallons of wastewater had passed through the column. The initial breakthrough is probably attributable to the inordinately high influent COD concentration at the beginning of the run as well as possible channeling or "short circuiting" at the incept because of the higher linear flow velocity. As in Test Series No. 2, a carbon capacity-volumetric throughput curve is developed for Test Series No. 3. This representation is shown in Figure 155.

Based on these relationships, the carbon capacity at a COD breakthrough level of 80 mg/l is approximately 0.38 lbs COD removed per lb carbon and the capacity at exhaustion is 0.48 lbs COD removed per lb carbon. This is higher than the value obtained from Test Series No. 2, but still slightly lower than indicated by the bench scale studies. It does indicate, however, a generalized basis for establishing a design carbon capacity in terms of COD removal which is necessary for sizing columns and estimating costs.

The color removal in the carbon column as a function of volume throughput is plotted in Figure 156. No significant breakthrough occurred during the test run. These results substantiate previous observations that the color-causative constituents are not resistant to adsorption and are easily removed by means of granular carbon columns.

TABLE 48

ACTIVATED CARBON COLUMN RESULTS - 4.0 gpm/ft^2 (Q = 28 gpm)

Sample No.	Date 1971	pH	COD mg/l	BOD mg/l	Color USPHS	Phenol mg/l	MBAS mg/l	TC mg/l	Gal/ day	Total Gal
601	7/22	2.3	412	160	677	5.6	4.9	120		
636		8.45	340	86	1,756	.2	4.4	104		
695										
601	7/23	2.5	416	173	903	5.4	4.8	126		[13,247]
636		7.4	269	94	1,058	.15	4.2	72	26,495	26,495
695		7.75	105	51.8	151	.05	.13	90		
601	7/24	2.55	311		483	5.6	3.4	150		[42,600]
636		7.1	184	60.6	504	.25	4.2	83	36,555	63,050
695		7.2	107	61.5		.05	.06	52		
601	7/25	2.55	444	206	729	4.15	2.8	144		[83.505]
636		6.85	142	29.0	484	.3	3.0	61	40,500	103.550
695		7.1	71	34.2	44	.05	.19	43		
601	7/26	2.85	356	125	915	.4	1.61	37		[129,500]
636		6.65	151	19.9	767	4.0	1.75	153	52,670	156,220
695			(98)							
601	7/27	2.90	395	106	684	4.35	2.09	105		[166,200]
636		7.05	163	17.5	672	.3	1.66	47	21,330	177,550
695		6.75	68		57	.05	.3	24		
601	7/28									[192,000]
636		7.1	110	10.8	718	.25	1.53	38	29,050	206,600
695		7.0	82	9.0	70	.1	.3	20		
601	7/29	9.8	410	198	896		2.66	114		[223,600]
636		6.95	131	13.2	592	.25		48	34,350	240,950
695		6.95	49	7.8	99	.1	3.4	18		
601	7/30	2.8	494	195	1,222	3.65	3.3	114		[255,450]
636		6.95	175	9.7	668	.3		48	29,050	270,000
695		7.0	82	6.3	38	.1	.3	20		
601	7/31	2.5	384	135	679	5.0	2.66			[286,000]
636		7.05	128	9.3	551	.2			32,260	302.260
695		7.05	62	6.0	116	.1	.43			
601	8/1	2.5	360	145	1,042	3.9	2.41			[317,300]
636		7.0	150	13.9	537	.35	2.04		30.460	332.720
695		7.0	85	7.8	90	.15	.43			
601	8/2	2.3	541	145	1.172	4.75	1.61	154		[348,020]
636		7.15	135	37	826	.2	1.57	51	30,600	363.320
695		7.15	83	12.5	95	.05	.48	25		
601	8/3	2.1	592	158	1,415	.35	1.5			[379,300]
636		6.8	294	24	1,317	6.25	1.45		32,530	395.850
695		7.0	109	10.8	119	.3	.54			
601	8/4	2.25	448	167	1.383	5.5	1.7	126		[412.300]
636		6.45	202	37	1,588	.4	1.26	74	33,500	429,700
695		7.2	73	7.5				22		
601	8/5	2.3	371	121	1.028	5.3		129		[446,400]
636		6.2	230	41	2,385	.35		68	35.200	464,600
695			(175)							
601	8/6	2.7	399	171	1,062	5.8		117		[483,600]
636		6.5	206	22	1,491	.35		57	38,300	502.900
695			(165)							
601	8/7	2.8	373	159		5.6		123		[518,400]
636		6.9	137	16.3		.35		34	33,800	536,700
695										
601	8/8	3.1	351	158		6.3		111		[557,100]
636		6.95	121	21		.35		38	40,800	577,500
695										
601	8/9	3.1	391	182		3.0		111		[596,000]
636		6.9	133	23		.2		48	36,900	o14,400
695										
601	8/10	2.9	326	152		9.1	1.41	105		[635,400]
636		7.1	171	19.2		.15	1.24	41	42,730	657,130
695										

601 = raw waste
636 = biological effluent (influent to carbon column)
695 = effluent from carbon column

( ) = average of 4-6 hr grab samples (filtered)
( )*= average of 4-6 hr grab samples (unfiltered)
[ ] = total volumetric throughput at midpoint of daily sampling period

Figure 153

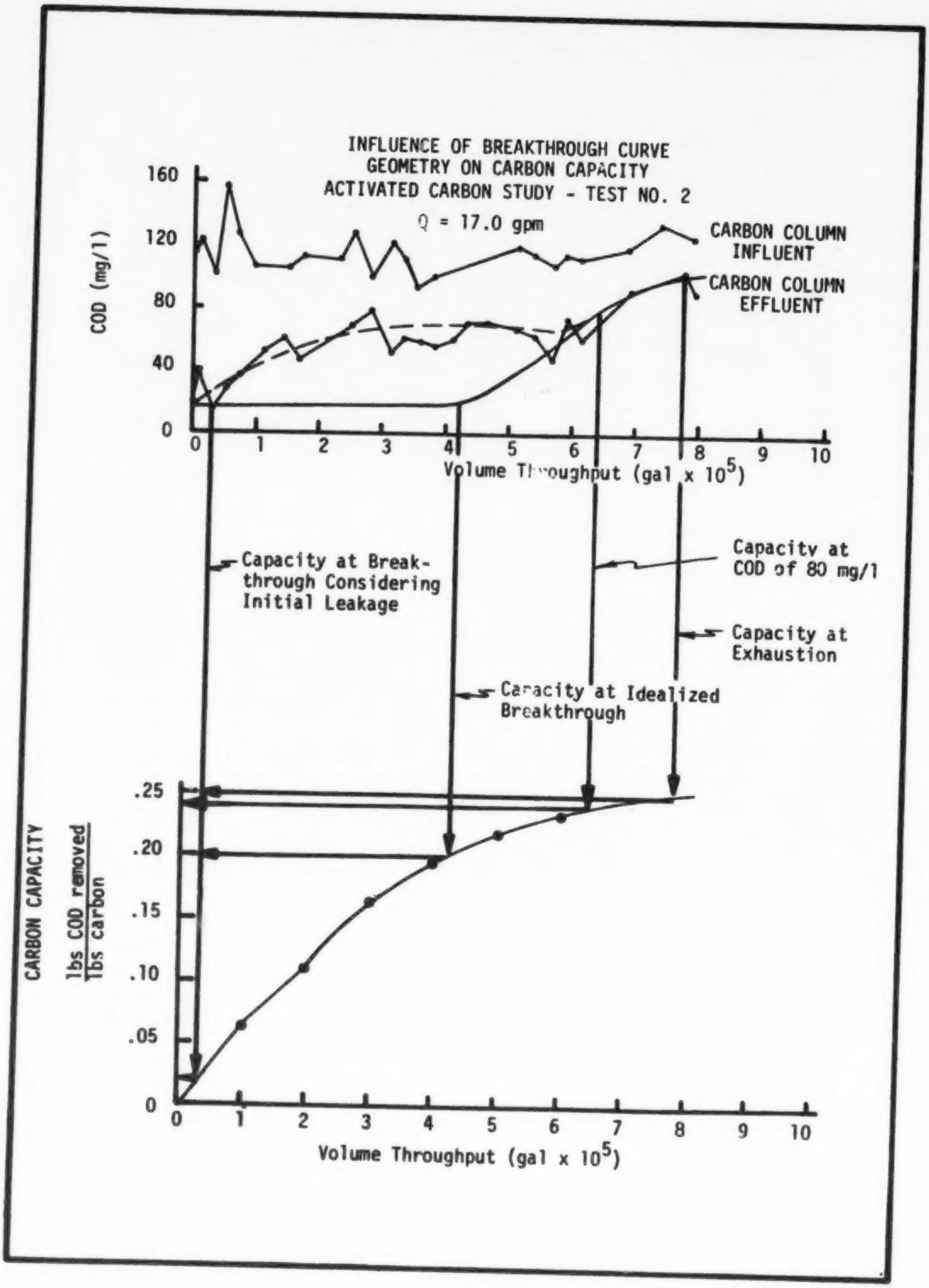

INFLUENCE OF BREAKTHROUGH CURVE
GEOMETRY ON CARBON CAPACITY
ACTIVATED CARBON STUDY - TEST NO. 2
Q = 17.0 gpm
COD (mg/l)
160
120
80
40
0
0
1
2
3
4
5
6
7
8
9
10
Volume Throughput (gal x 10^5)
CARBON COLUMN INFLUENT
CARBON COLUMN EFFLUENT
Capacity at Breakthrough Considering Initial Leakage
Capacity at COD of 80 mg/l
Capacity at Exhaustion
Capacity at Idealized Breakthrough
CARBON CAPACITY
lbs COD removed / lbs carbon
.25
.20
.15
.10
.05
0
0
1
2
3
4
5
6
7
8
9
10
Volume Throughput (gal x 10^5)

Figure 154

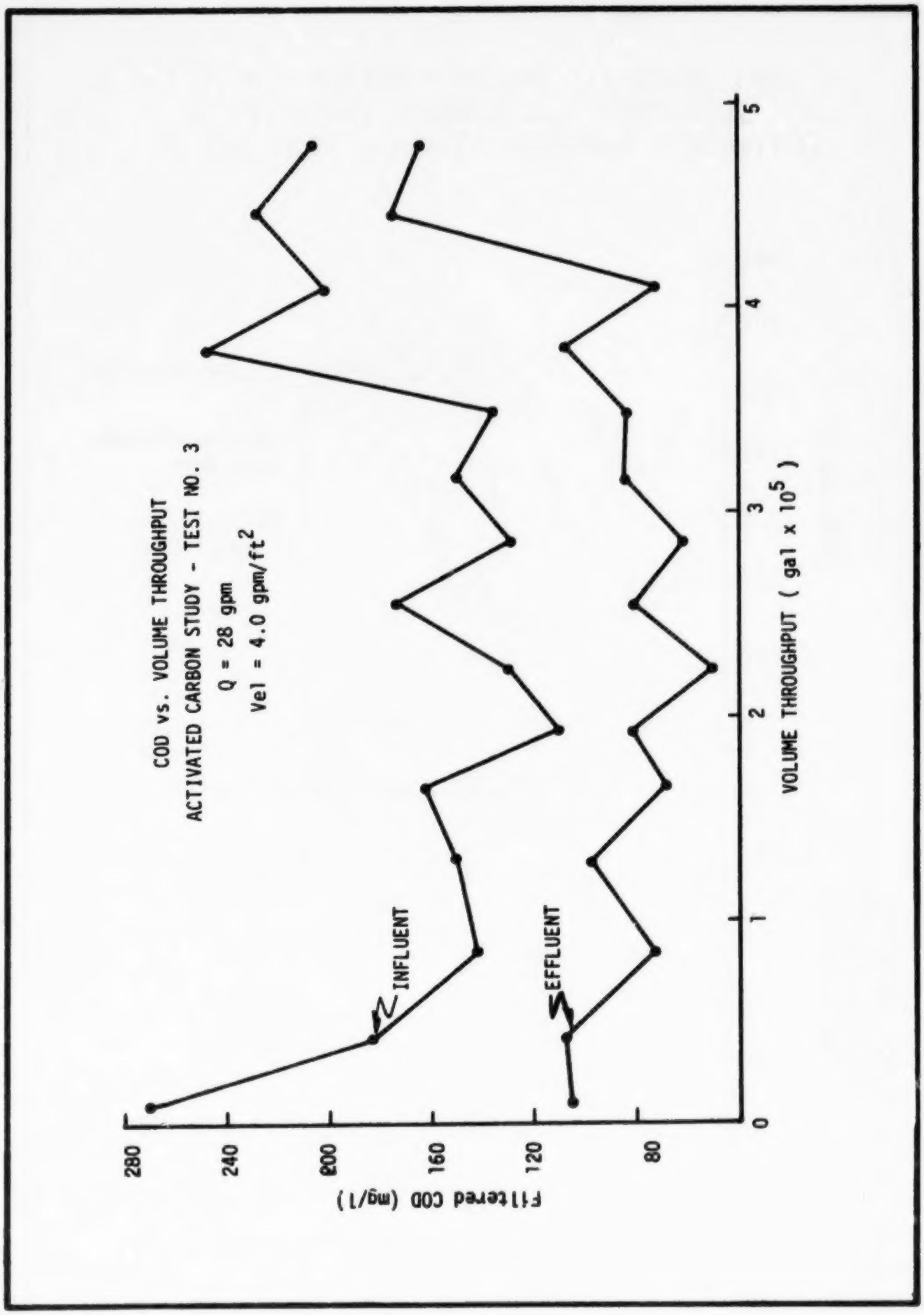

COD vs. VOLUME THROUGHPUT
ACTIVATED CARBON STUDY - TEST NO. 3
Q = 28 gpm
Vel = 4.0 gpm/ft^2
INFLUENT
EFFLUENT
Filtered COD (mg/l)
280
240
200
160
120
80
0
1
2
3
4
5
VOLUME THROUGHPUT (gal x 10^5)

Figure 155

INFLUENCE OF BREAKTHROUGH CURVE GEOMETRY ON CARBON CAPACITY ACTIVATED CARBON STUDY—TEST NO. 3

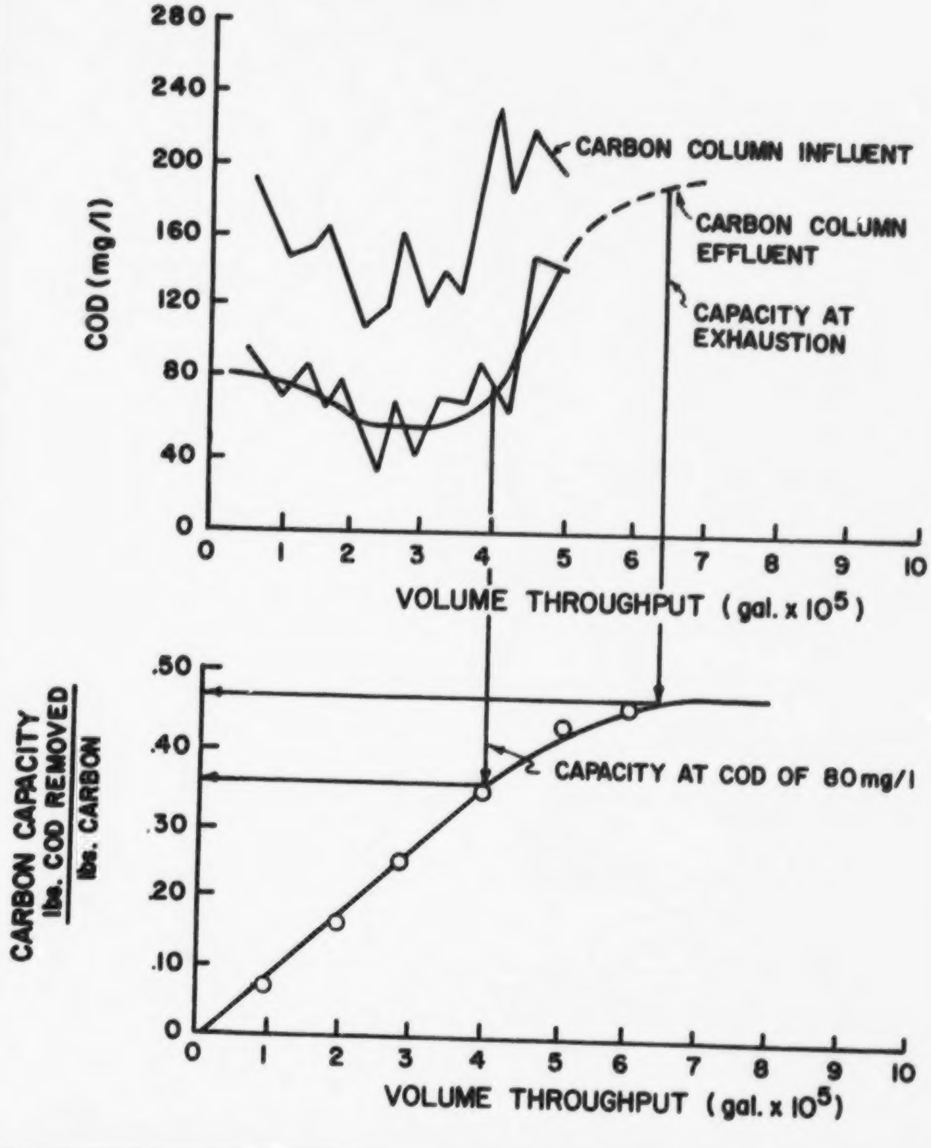
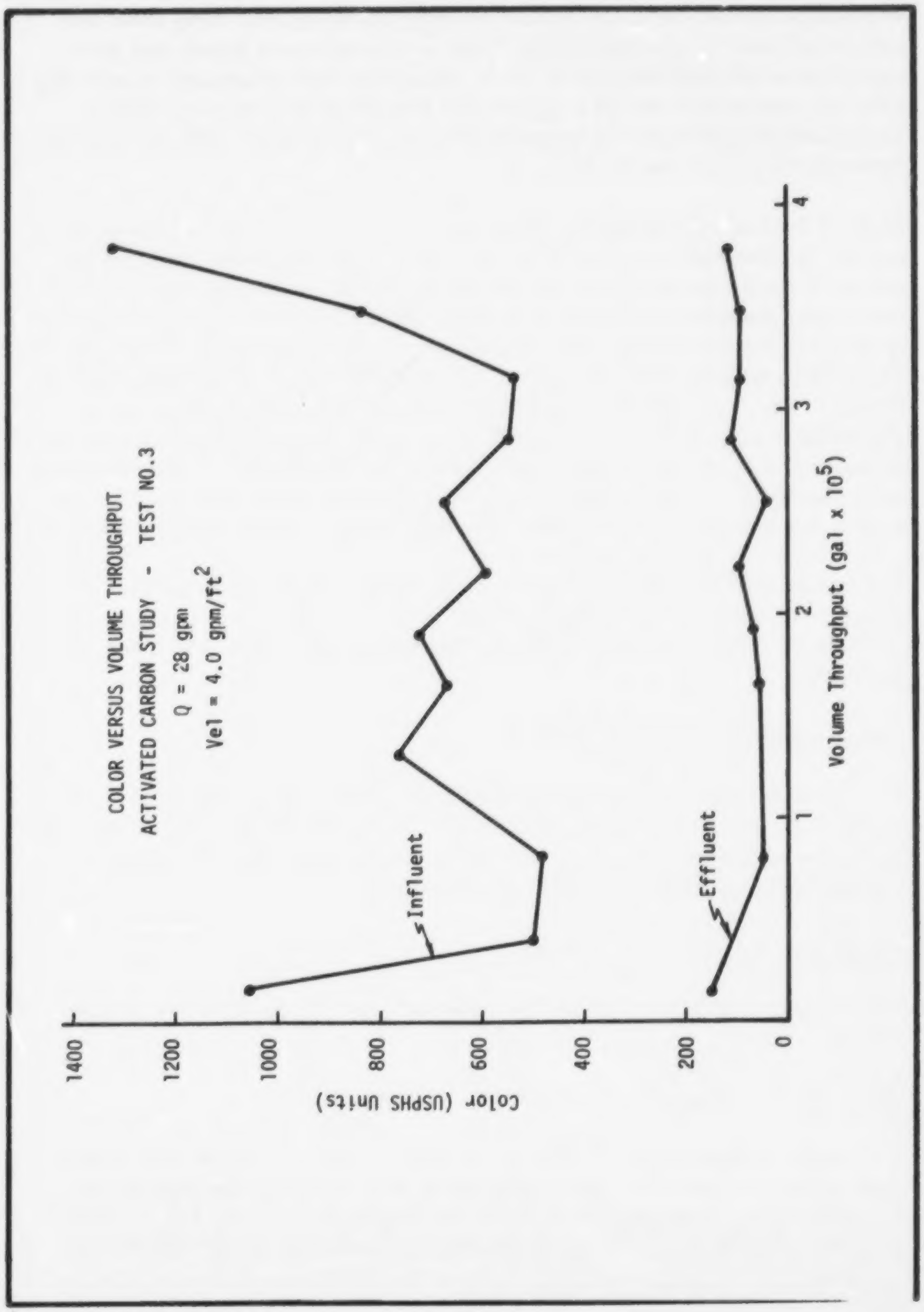


Figure 156


COLOR VERSUS VOLUME THROUGHPUT
ACTIVATED CARBON STUDY - TEST NO.3
Q = 28 gpm
Vel = 4.0 gpm/ft²
Influent
Effluent
Color (USPHS Units)
0
200
400
600
800
1000
1200
1400
Volume Throughput (gal x 10⁵)
1
2
3
4

Summary:

The design criteria for a conceptual effluent polishing step using fixed bed carbon columns can be established based on the extensive bench and pilot scale studies as reported herein. It is recognized that subsequent events may alter the design basis to some extent, but the information as presented is considered adequate for the purposes of preliminary design, effluent quality determination, and cost estimation.

Most of the carbon treatability reported in this Chapter in terms of organic quality has been presented in terms of COD. This is justified based on the nature and reproducibility of the analytical procedure as compared to BOD. Moreover, the relationship between BOD and COD for the biological-carbon systems for similar wastewaters has been previously documented (Reference 12). This relationship as shown in Figure 157 indicates that at an effluent COD of 80 mg/l, which is possible to obtain through the biological-carbon system, the effluent BOD will be less than 15 mg/l during summer operating conditions as confirmed by the data presented in Tables 46 through 48. These levels are not expected to increase significantly during winter operations, and in any event, are expected to satisfy the "override" criteria as set forth by DRBC.

The results from the bench and pilot scale studies which influence the conceptual design of the carbon effluent polishing system are summarized in Table 49. Based on these numbers, the following criteria are selected for design:

Design Linear Velocity - 8 gpm/ft^2

This is a higher flow rate than applied to the pilot scale column, but it is within the range of the bench scale tests. This application rate will provide for higher carbon utilization as well as enhanced operation with respect to TSS removal and backwash cycle requirements.

Design Contact Time - 20 minutes

This contact time is justified on the observed bench and pilot scale column studies.

Design Carbon Capacity - 0.40 lbs COD removed/lb carbon

It is observed from Table 49 that the carbon capacity increases with linear flow velocity. The selected capacity of 0.40 is based on the pilot scale Test Series No. 3 properly weighted with respect to a higher design linear velocity and a capacity for a pre-selected breakthrough of 80 mg/l COD.

Figure 157

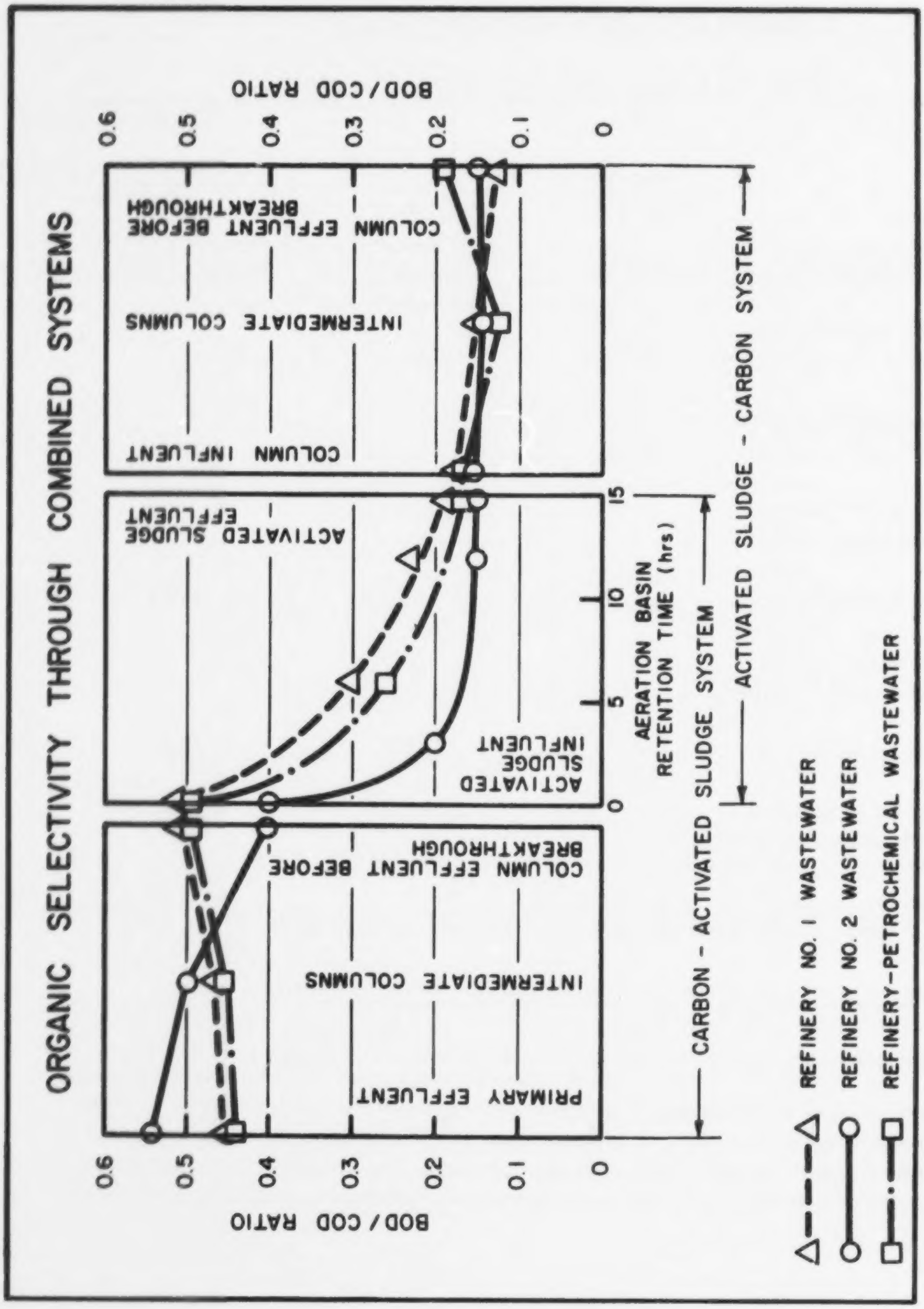

ORGANIC SELECTIVITY THROUGH COMBINED SYSTEMS
BOD / COD RATIO
0.6
0.5
0.4
0.3
0.2
0.1
0
PRIMARY EFFLUENT
INTERMEDIATE COLUMNS
COLUMN EFFLUENT BEFORE BREAKTHROUGH
ACTIVATED SLUDGE INFLUENT
ACTIVATED SLUDGE EFFLUENT
COLUMN INFLUENT
INTERMEDIATE COLUMNS
COLUMN EFFLUENT BEFORE BREAKTHROUGH
0
5
10
15
AERATION BASIN RETENTION TIME (hrs)
CARBON - ACTIVATED SLUDGE SYSTEM
ACTIVATED SLUDGE - CARBON SYSTEM
REFINERY NO. 1 WASTEWATER
REFINERY NO. 2 WASTEWATER
REFINERY-PETROCHEMICAL WASTEWATER

TABLE 49

SUMMARY OF CARBON CAPACITY VALUES

BENCH AND PILOT SCALE CARBON COLUMNS

TEST DESCRIPTION	Linear Flow Velocity (gpm/ft^2)	Contact Time (min.)	Carbon Capacity (lbs COD removed / lb carbon) @ breakthrough 80 mg/l COD	@ exhaustion
1. Bench scale columns 2.9" x 6' - downflow	4.4	18.8	-	0.5
2. Bench scale columns 2.9" x 6' - downflow	9.8	8.7	-	0.70
3. Pilot scale column 3' diameter - upflow				
Test Series No. 1	3.1	17.8	0.19	0.20
4. Pilot scale column 3' diameter - upflow				
Test Series No. 2	2.4	23.0	0.23	0.25
5. Pilot scale column 3' diameter- upflow				
Test Series No. 3	4.0	14.0	0.38	0.48

NOTE: Carbon in all cases was granular "Nuchar" 12 x 40 mesh

Conventional Biological Treatment Using Powdered Activated Carbon

The direct application of powdered carbon to the activated sludge aeration basin has been the subject of investigation for several years, particularly where effluent color and residual organics are in question. Because of the circumstances inherent in the overall study as related to effluent quality, it was determined to evaluate this approach from the standpoint of organic removal and solids-liquid separation. The disposal or reconditioning of the sludge-carbon mixture was not included in the scope of the field investigations, although this facet most probably represents the critical path.

Procedure:

A direct comparison approach was taken in evaluating this system by operating two parallel biological systems simultaneously; namely, one with powdered carbon addition and one without. Each of these systems had a feed rate of 25 gpm, an aeration detention time of 12 hours and operated under identical environmental conditions. Carbon addition to aeration tank "B" was accomplished by feeding a powdered carbon slurry to the tank through a time controlled, air operated, three-way ball valve arrangement such that a predetermined feed rate was continuously applied to the system. The MLSS concentration of the powdered carbon system ranged from 2,000 mg/l to 6,000 mg/l while the MLSS concentration of the conventional system was controlled at approximately 2,000 mg/l. The performance of each of the systems was monitored daily in terms or organic and color removal and sludge settleability.

Results:

The daily results were grouped and summarized over identical operational periods. This summary is presented in Table 50. As expected, the powdered carbon system in terms of both organic removal and color removal outperformed the conventional biological system. Both systems when tested during the winter months, however, exhibited relatively low organic removal efficiencies. Additionally, the sludge settleability in terms of the SVI for the powdered carbon system was markedly lower than that of the conventional system.

Since both the conventional biological system and the powdered carbon system were operated concurrently, the environmental conditions affecting both systems were essentially normalized when considering the comparative performance of the two systems. Thus, an estimation of the effects of the carbon dosage could be obtained in terms of additional removal efficiency. Figure 158 presents the observed additional BOD-COD removed with respect to the powdered carbon dosage. Figure 159 presents the effluent color results as a function of carbon dosage.

Summary:

As evidenced by the pilot plant data presented, the addition of powdered carbon to the aeration basin enhances the overall removal efficiencies of the biological system. Moreover, sludge settleability and

TABLE 50

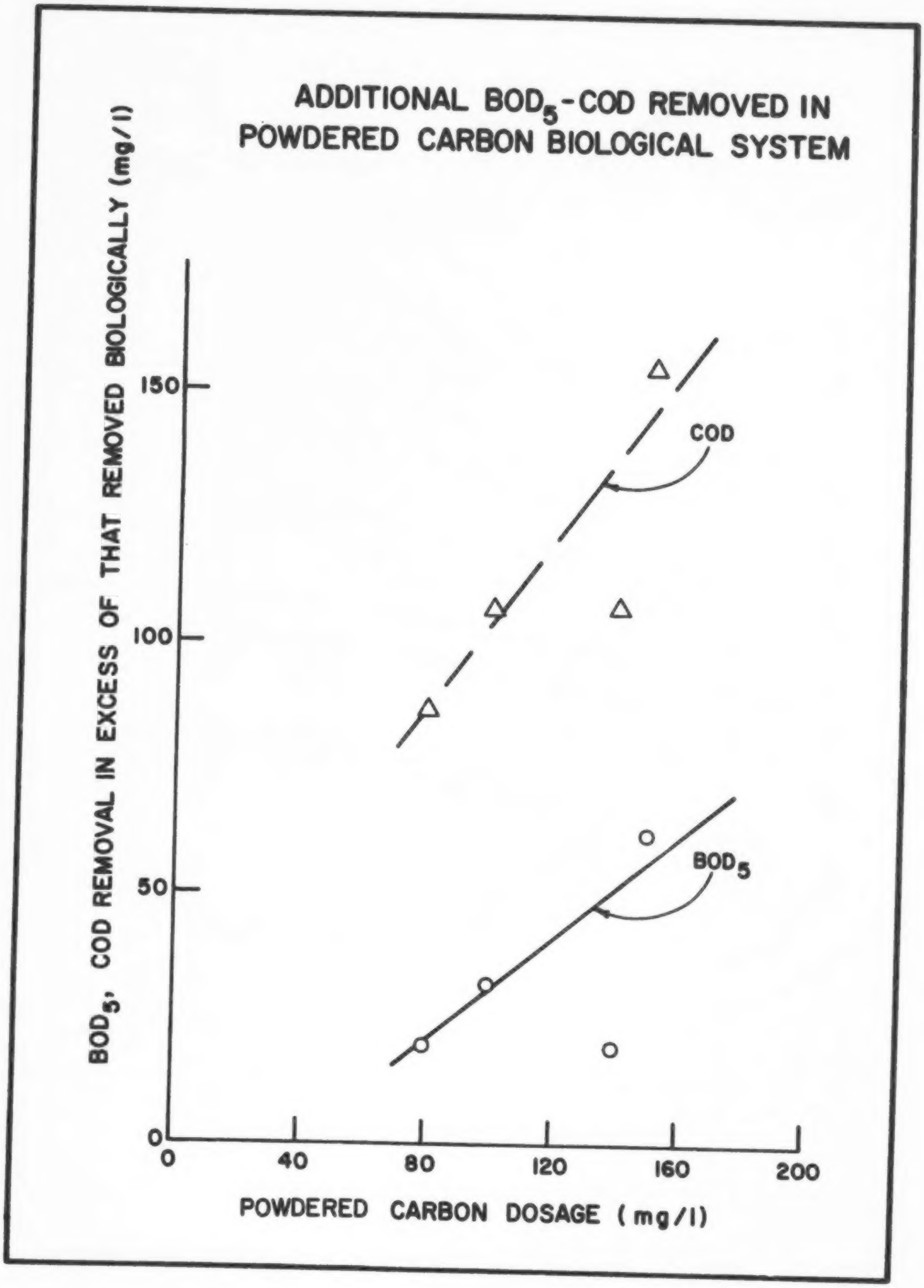
RESULTS OF THE CONVENTIONAL AND CARBON ACTIVATED SLUDGE SYSTEMS

TEST PERIOD	INF. BOD* mg/l	EFF. BOD* mg/l	PERCENT REMOVAL	INF. COD* mg/l	EFF. COD* mg/l	PERCENT REMOVAL	INF.** COLOR	EFF.** COLOR	SVI	POWDERED CARBON FEED RATE mg/l
CONVENTIONAL BIOLOGICAL SYSTEM										
I	141	56	58	507	269	48	620	570	48	
II	192	81	57	615	321	48	1,062	995	49	
III	144	76	46	533	301	43	860	759	46	
IV	193	128	34	622	394	37	956	925	47	
BIOLOGICAL SYSTEM WITH POWDERED CARBON ADDITION										
I	141	36	73	507	162	68	620	87	26	140
II	192	61	68	615	234	61	1,062	513	30	80
III	144	45	59	533	195	63	860	273	28	100
IV	193	65	66	622	240	61	956	262	29	150

* BOD and COD results based on soluble organics

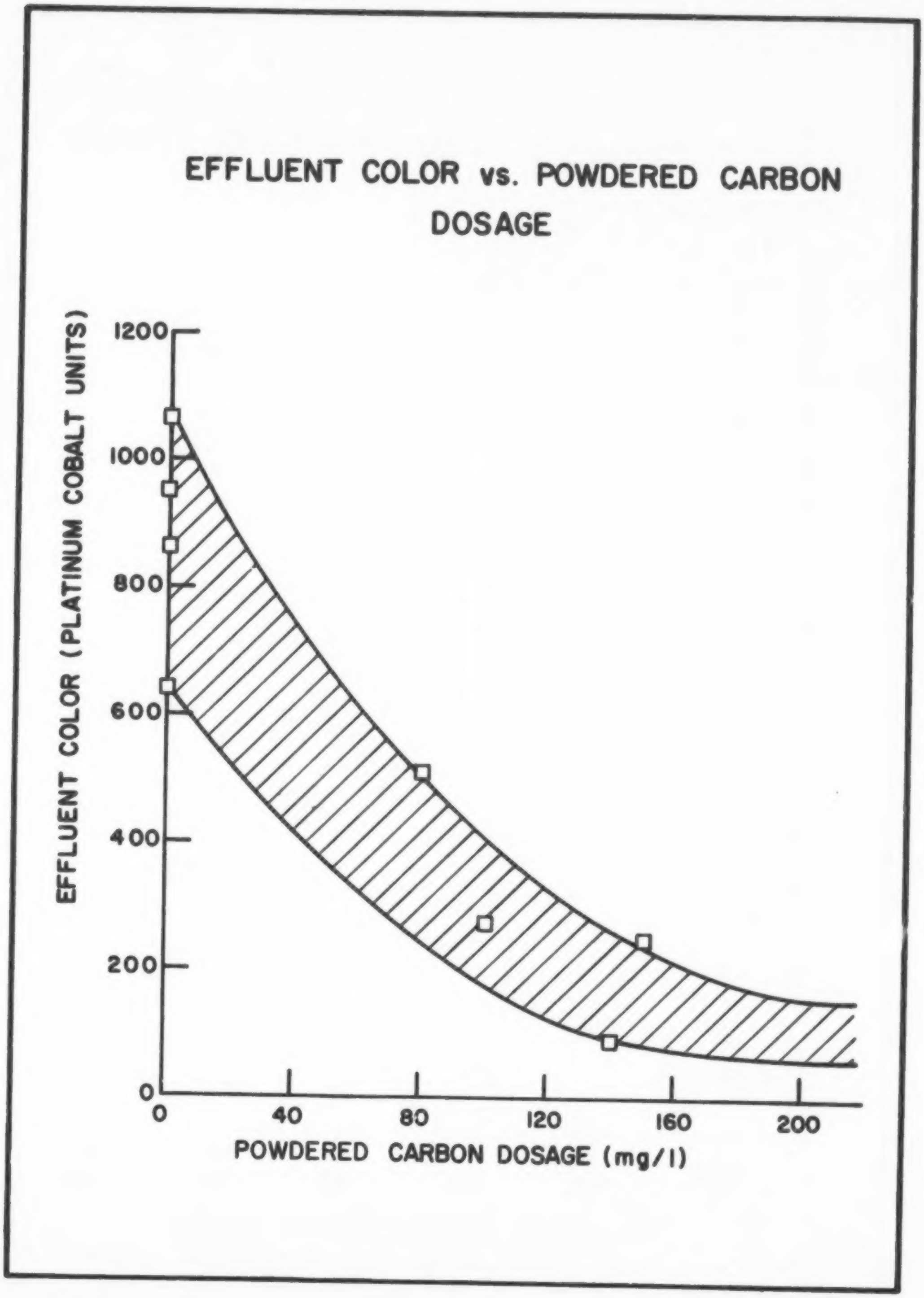
** Platinum cobalt units

Figure 158



ADDITIONAL BOD5-COD REMOVED IN POWDERED CARBON BIOLOGICAL SYSTEM
BOD5, COD REMOVAL IN EXCESS OF THAT REMOVED BIOLOGICALLY (mg/l)
150
100
50
0
COD
BOD5
0
40
80
120
160
200
POWDERED CARBON DOSAGE (mg/l)

Figure 159


EFFLUENT COLOR vs. POWDERED CARBON DOSAGE
EFFLUENT COLOR (PLATINUM COBALT UNITS)
1200
1000
800
600
400
200
0
0
40
80
120
160
200
POWDERED CARBON DOSAGE (mg/l)

color removal are improved. Based on the results of this test, a carbon dosage in excess of 150 mg/l would be required in order to satisfy the effluent color regulations of 100 units. At this anticipated feed rate, the spent carbon-biological sludge would necessarily have to be regenerated to economically compete with alternate color removal systems. The critical path of this system's applicability, therefore, is the sludge handling phase of the treatment cycle. The sludge, containing spent carbon, excess biological mass, and other particulates, must be segregated and the carbon recovered in an efficient and economical manner. Since this type of regeneration and recovery has not been attempted on a large scale basis, a forceful recommendation of the system cannot be made at this time. Additionally, effluent filtration may be necessary as a tertiary step to this process for the elimination of carbon fines.

Upflow Sand Filtration

Sand filtration was demonstrated as an effluent polishing process with the three foot in diameter upflow filter as described in the Pilot Scale Activated Carbon Test section of this Section. The filter media gradation from bottom to top is described as follows:

> 2.5 cubic feet of 1 1/4" x 1 1/2" gravel, six cubic feet of 3/8" x 5/8" gravel, seven cubic feet of 2-3 mm sand and 40 cubic feet of 1-2 mm sand. The filter was operated at three hydraulic loadings of four, six, and eight gpm/ft^2 utilizing the effluent from the final clarifier of the biological pilot plant.

Procedure

The operational procedures used for each filter run are listed below:

1. The filter was backwashed prior to each test run. The backwash cycle included bumping the filter with 30 cfm of air for three to four minutes. The 100 gpm backwash rate was then continued for an additional six to 10 minutes until a clear effluent was produced.

2. The filtration cycle was initiated. The hydraulic flow rate was controlled manually with a valve.

3. Turbidity tests were performed on grab samples of the effluent throughout the filter run. The break point was established when the turbidity reached a pre-defined level.

Results

The data from a typical filter run is presented in Figure 160. The turbidity remained reasonably constant throughout the filter run until the actual breakthrough occurred. Organic removal in terms of COD in the run presented here increased as breakthrough was approached. However, the organic removal was minimal across the filter. Additionally, no color removal was observed during any of the filter tests.

The removal of suspended solids as a function of the hydraulic loading is presented in Figure 161. As noted, the total solids accumulated in the sand media at breakthrough decreased with increased hydraulic loading. However, since the quality of the effluent from each of the hydraulic loadings was essentially the same with respect to COD and TSS, the design hydraulic loading should be based on filter service time and backwash frequency.

Summary

The results of the filtration studies cited here indicate that only minimal residual organic and color removal can be expected through the filter. This is reasonable when considering that most organics removed by filtration are of a colloidal and suspended nature and the residual organic constituents of the combined waste are primarily soluble. Based on these results, filtration does not appear to be technically justified on the basis of effluent quality regulations.

Microstraining Pilot Studies

Microstraining pilot studies were conducted as an effluent polishing process with a Micro-Matic straining system four feet in diameter and two feet wide. The strainer was fabricated with 12 stainless steel straining assemblies with a total area of 24 square feet. The water entered the center of the rotating drum, flowed through the screens and out the effluent weir box. As the drum rotated, the screens were backwashed by means of a spray system located on the top side of the drum.

Procedure

The procedure followed for each of two test runs entailed pumping the biological effluent through the straining system while monitoring the influent and effluent suspended solids. Two separate tests were conducted, the first at a flow rate of 21 gpm with 24 square feet of filter area, and the second at a flow rate of 43.5 gpm with 12 square feet of filter area.

TYPICAL RESULTS FOR
PILOT UPFLOW SAND FILTER EXPERIMENTS

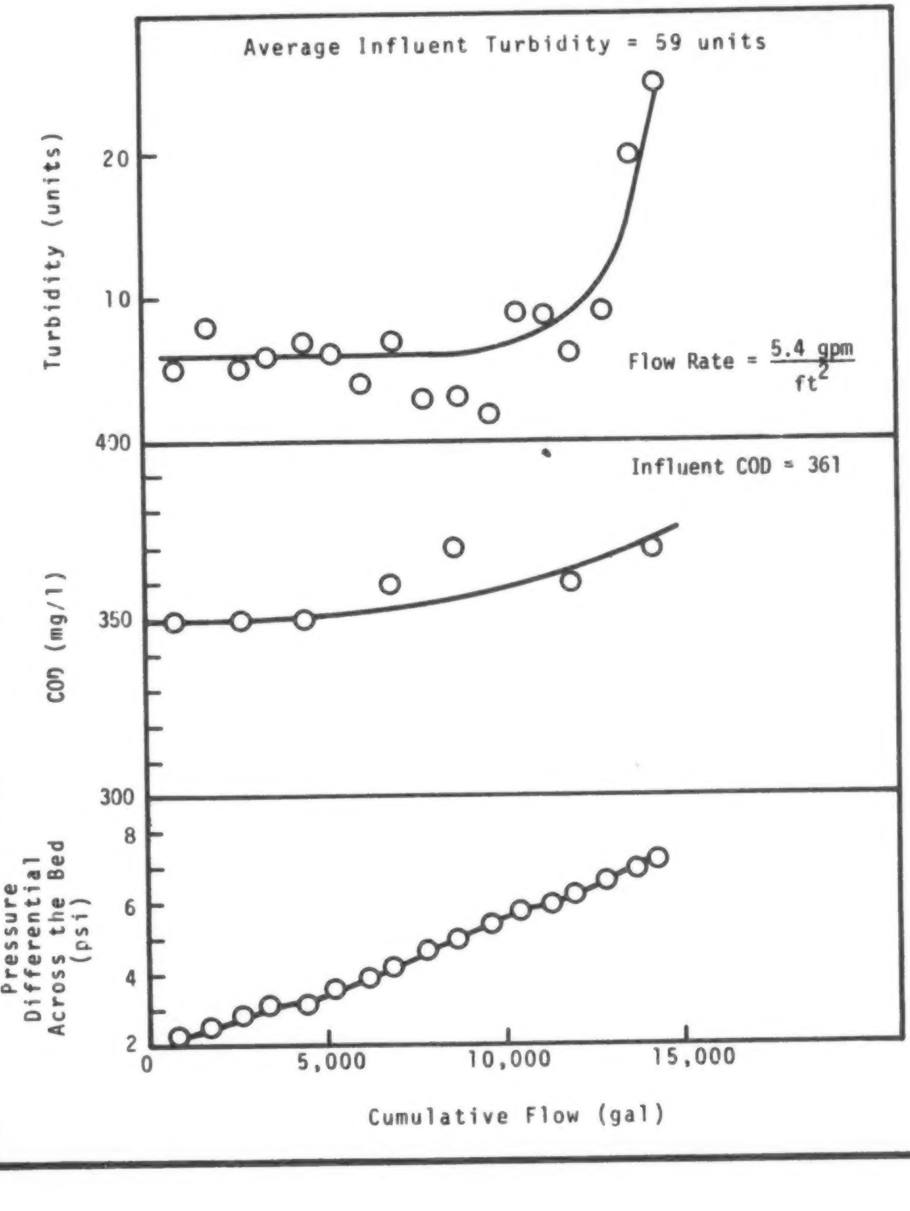
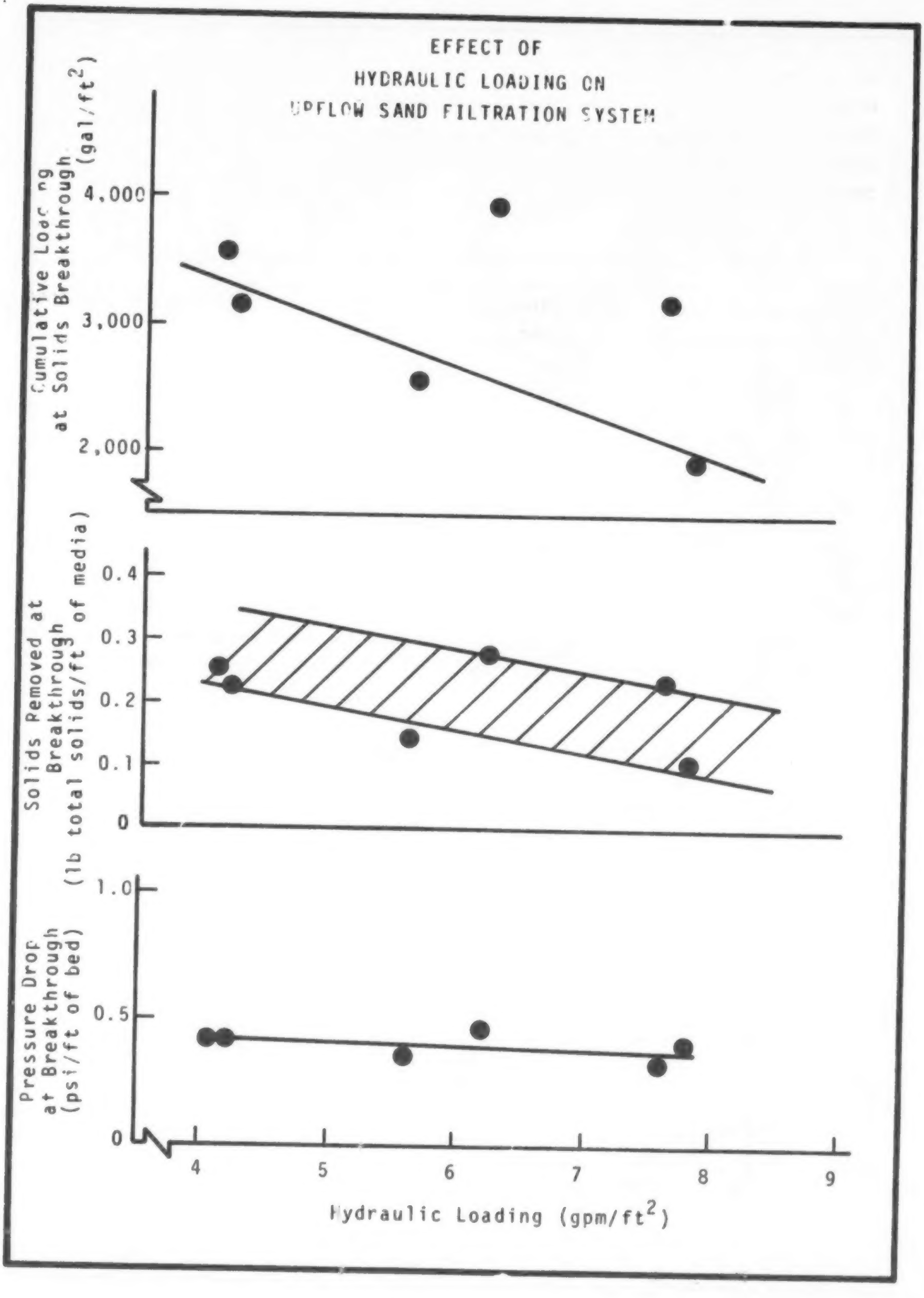


Figure 161



EFFECT OF
HYDRAULIC LOADING ON
UPFLOW SAND FILTRATION SYSTEM
Cumulative Loading at Solids Breakthrough (gal/ft^2)
4,000
3,000
2,000
Solids Removed at Breakthrough (lb total solids/ft^3 of media)
0.4
0.3
0.2
0.1
0
Pressure Drop at Breakthrough (psi/ft of bed)
1.0
0.5
0
4
5
6
7
8
9
Hydraulic Loading (gpm/ft^2)

Results

The results of the two test runs are tabulated in Table 51. As noted, the suspended solids removal efficiency was low during both tests although somewhat better solids removal was experienced at the higher flow rate. Since the suspended solids from a biological system are quite small, the 20 micron steel mesh screens were apparently too large to adequately entrap the suspended solids.

Summary

The use of a microstraining system does not appear to be technically justified in this particular application based on the pilot scale studies.

TABLE 51

MICROSTRAINING RESULTS *

Results At 21 gpm With 24 ft^2 Screen Area

Accumulated Gallons	Influent TSS mg/l	Effluent TSS mg/l
5,000	60	50
10,000	40	35
15,000	50	40
20,000	60	50

Results At 43.5 gpm With 12 ft^2 Screen Area

Accumulated Gallons	Influent TSS mg/l	Effluent TSS mg/l
5,000	80	70
10,000	70	60
15,000	60	45
20,000	70	55
25,000	50	40

* Samples based on grab type samples - results are tabulated as mean values.

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SECTION VII

CONCEPTUAL DESIGN AND TREATMENT COST ESTIMATES

The conceptual design and subsequent cost estimates of the regional treatment facility are presented in this Section. The basis for selection of the most appropriate unit processes to be included in the optimal treatment system was predicated on economic considerations, process applicability and reliability as determined by the bench and pilot studies, and the effluent quality and stream objectives of the Delaware River Basin Commission. The treatment system developed was based on the current flow estimates totaling 72 MGD and on the raw wastewater characterization data as presented in Section IV of this report. It has been determined that the proposed system will meet the necessary effluent criteria as presented in Section VIII.

The major treatment processes selected include an activated sludge system followed by an activated carbon effluent polishing system. Pretreatment processes include neutralization followed by primary clarification. Additionally, sludge digestion and sludge dewatering processes were selected to handle both the primary and wasted activated sludges. A schematic of the proposed treatment system is presented in Figure 5.62.

DESIGN CALCULATIONS AND COST ESTIMATES

The design criteria, design calculations, and cost estimates for the major unit processes are included herein. The design criteria as presented are based on the results of the pilot and bench studies as discussed in Section VI. The estimated costs are based on an ENR index of 1400 to be consistent with estimates cited in the Preliminary Engineering Report and the Interim Pilot Plant Report.

Neutralization

The proposed neutralization system includes a premixing basin prior to a series of four two-stage neutralization basins. Dolomitic quick lime will be slaked and added to the appropriate basin as required with a pH controlled feed mechanism. As this system is necessary for only two of the industrial participants, namely, the duPont Chambers Works and the duPont Carney's Point Plant, some of the costs will be borne directly by these two participants. However, the combined premixing-neutralization process is designed to act as a disinfection process as well, utilizing the available acid as a biocide. Based on the pilot plant data,

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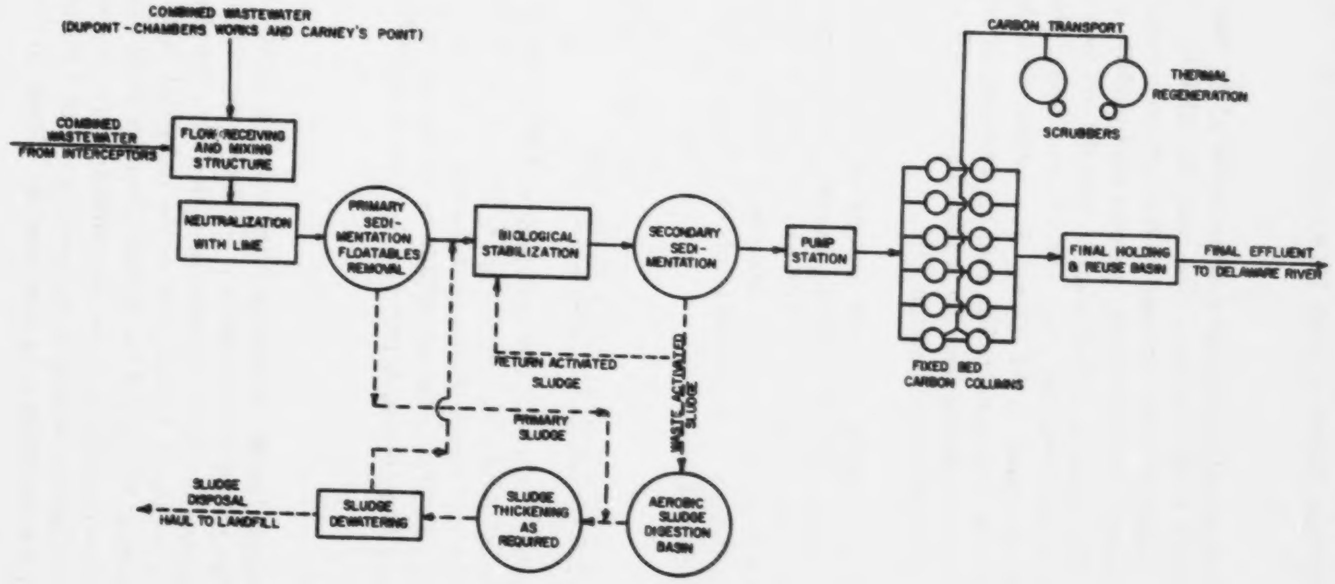
Figure 162

SCHEMATIC OF PROPOSED ACTIVATED SLUDGE - CARBON ADSORPTION TREATMENT SYSTEM


COMBINED WASTEWATER (DUPONT - CHAMBERS WORKS AND CARNEY'S POINT)
COMBINED WASTEWATER FROM INTERCEPTORS
FLOW-RECEIVING AND MIXING STRUCTURE
NEUTRALIZATION WITH LIME
PRIMARY SEDI-MENTATION FLOATABLES REMOVAL
BIOLOGICAL STABILIZATION
SECONDARY SEDI-MENTATION
PUMP STATION
CARBON TRANSPORT
SCRUBBERS
THERMAL REGENERATION
FIXED BED CARBON COLUMNS
FINAL HOLDING & REUSE BASIN
FINAL EFFLUENT TO DELAWARE RIVER
RETURN ACTIVATED SLUDGE
WASTE ACTIVATED SLUDGE
PRIMARY SLUDGE
AEROBIC SLUDGE DIGESTION BASIN
SLUDGE THICKENING AS REQUIRED
SLUDGE DEWATERING
SLUDGE DISPOSAL HAUL TO LANDFILL

this arrangement will be effective as no fecal coliforms were ever observed in the influent or effluent during the entire pilot plant study. Therefore, the cost as presented includes the capital and operating costs of the basins alone and do not reflect the costs of lime addition or storage facilities. Moreover, only the basin sizing and power requirements are presented herein.

Process Requirements

Premixing Basin

Flow = 72 MGD
Detention Time = 15 minutes
Power Level for Complete Mixing - 0.4 HP/1,000 gal

Neutralization Basins

Flow = 72 MGD
Number of Basins = 4 two-stage
Detention Time/Stage = 15 minutes
Power Level for Complete Mixing = 0.4 HP/1,000 gal

Design Calculations

Premixing Basin

Calculate basin size using a 15-minute detention time:

$$\text{Basin Size} = \frac{(72 \times 10^6 \text{ gal/day})(15 \text{ min})}{(1{,}440 \text{ min/day})(7.48 \text{ gal/ft}^3)} = 100{,}000 \text{ ft}^3$$

Calculate basin area assuming 12 ft depth with a square configuration:

$$\text{Basin Area} = \frac{100{,}000 \text{ ft}^3}{12} = 8{,}340 \text{ ft}^2$$

$$\text{Length} = \text{Width} = \sqrt{8{,}340 \text{ ft}^2} = 91.3 \text{ ft} \qquad \text{USE 100 ft}$$

Calculate power requirements @ 0.4 HP/1,000 gal:

$$\text{Total HP} = \frac{(12 \text{ ft})(100 \text{ ft})(100 \text{ ft})(7.48 \text{ gal/ft}^3)(0.4 \text{ HP})}{1{,}000 \text{ gal}} \approx 360$$

Use four 100 HP slow speed mixers on 50 ft centers

Neutralizing Basins

Calculate Volume of each stage using a detention time of 15 minutes:

$$\text{Volume/Stage} = \frac{(72 \times 10^6 \text{ gal/day})\ (15 \text{ min})}{(4 \text{ systems})\ (1{,}440 \text{ min/day})} = 187{,}500 \text{ gal}$$

Calculate basin area assuming 12 ft depth with a square configuration:

$$\text{Area} = \frac{187{,}500 \text{ gal}}{(7.48 \text{ gal/ft}^3)\ (12 \text{ ft})} = 2{,}080 \text{ ft}^2$$

$\text{Length} = \text{Width} = \sqrt{2{,}080 \text{ ft}^2} = 46 \text{ ft}$ USE 50 feet

Calculate power requirements @ 0.4 HP/1,000 gal

$$\text{HP/stage} = \frac{(12 \text{ ft})\ (50 \text{ ft})\ (50 \text{ ft})\ (7.48 \text{ gal/ft}^3)\ (0.4 \text{ HP})}{1{,}000 \text{ gal}}$$

HP/stage = 90 Use one 100 HP slow speed mixer per basin

Design Summary

Premixing Basin

Basin Dimensions = 12 ft x 100 ft x 100 ft
Power Requirements = 4-100 HP mixers

Neutralization Basins

Number of Basins = 4 two-stage basins
Basin Dimensions/Stage = 12 ft x 50 ft x 50 ft
Power Requirements = 8-100 HP mixers -- one each stage

Cost Estimate*

Item	Est. Cost	
Concrete and Earthwork	$390,000	
Mechanical (Mixers)	216,000	
Electrical	18,000	
Piping and Valves	45,000	
Structural	40,000	
Basin Lining	50,000	
Contingencies and Miscellaneous	51,000	
Total Capital		$810,000

Operating Costs		165,000	
Fixed Annual Costs		58,000	
	Total Annual		$223,000

* Based on apportioned costs only as described above.

Primary Clarification

The proposed primary clarification system includes 12 parallel basins equipped with mechanical sludge removal mechanisms. Sludge pumps are provided for solids removal to the dewatering process. Each basin will have two parallel flight assemblies designed for both sludge and scum collection.

Process Requirements

Flow = 72 MGD
Number of Basins = 12 rectangular shaped
Detention Time =>two hours
Overflow Rate (not to exceed) = 800 gal/day/ft^2
Sludge Production = 2,000 lbs 10^6 gal @ one percent concentration
(Based on pilot plant observations)

Design Calculations

Calculate surface area per basin assuming an average SWD of 10 feet:

$$\text{Surface Area} = \frac{(72 \times 10^6 \text{ gal/day})}{(12 \text{ basins})(800 \text{ gal/day/ft}^2)}$$

Surface Area = 7,500 ft^2

Calculate basin length using a maximum width of 40 feet:

$$\text{Basin Length} = \frac{7{,}500 \text{ ft}^2}{40 \text{ ft}} = 187 \text{ ft}$$

Use 200 ft basins to allow for weir location.

Check detention against a minimum of two hours:

$$\text{Volume per basin} = (10 \text{ ft})(40 \text{ ft})(200 \text{ ft})(7.48 \text{ gal/ft}^2) = 600{,}000 \text{ gal}$$

$$\text{Detention Time} = \frac{(600{,}000 \text{ gal})(24 \text{ hrs/day})}{6.0 \times 10^6 \text{ gal/day}} = 2.4 \text{ hours}$$

Therefore the detention time is adequate.

Calculate sludge pumping requirements assuming continuous removal at one percent solids content:

$$\text{Volume of Sludge} = \frac{(2{,}000\ \text{lbs}/10^6\ \text{gal})\ (72\ \text{MGD})\ (10^6\ \text{gal})}{(10{,}000\ \text{mg/l})\ (8.34\ \text{lbs/gal})}$$

Volume of Sludge = 1,730,000 gal/day

Use four 600 gpm pumps - two operational and two standby. Located in a centralized pump station.

Design Summary

Number of clarifiers = 12

Basin Dimensions = 10 ft SWD x 40 ft x 200 ft

Pumps = four 600 gpm

Sludge Removal Mechanisms = 24 - 20 ft flight assemblies (two each basin)

Cost Estimate

Item	Est. Cost	
Concrete & Earthwork	$1,565,000	
Mechanical (pumps & flight assemblies)	876,000	
Electrical	15,000	
Piping & Valves	190,000	
Instrumentation and Controls	23,000	
Hand Rails	58,000	
Contingencies & Miscellaneous	363,000	
Total Capital		$3,090,000
Operating Costs	$120,000	
Fixed Annual Cost	221,500	
Total Annual		$341,500

Secondary Biological System

The conceptual design of the secondary biological system includes six completely mixed aeration basins followed by 12 center-fed circular clarifiers. Three communal pump stations are provided for returning the activated sludge to the aeration basins.

Process Requirements

Aeration System:

Flow = 72 MGD

Aeration Detention Time = 12 hours (based on maximum conditions during the summer, see Section VI)

Oxygen Utilization = 2,164 lbs $O_2/10^6$ gal (based on maximum conditions during the winter, see Section VI)

Aeration Transfer Efficiency = 2.9 lbs O_2/HP-hr (see Section VI)
Power Level for Complete Mixing = 0.15 HP/1,000 gal
Sludge Production = 500 lbs/10^6 gal (see Section VI)

Final Clarification:

Flow = 72 MGD

Overflow Rate (not to exceed) = 700 gal/day/ft^2

Theoretical Detention Time = >two hours

Sludge Return = 50% with 75% possible

Sludge Concentration = one to two percent

Design Calculations

Aeration Basins:

Calculate basin surface area assuming six basins with a depth of 12 feet:

$$\text{Surface Area} = \frac{(36 \times 10^6 \text{ gal})}{(6 \text{ basins})(7.48 \text{ gal/ft}^3)(12 \text{ ft})} = 66{,}800 \text{ ft}^2$$

Based on aeration requirements as tabulated below, calculate basin dimensions using ten 100 HP aerators per basin at a power level of 0.15 HP/1,000 gal. Calculate square surface mixing area of each aerator assuming a basin depth of 12 feet.

$$\text{Surface Area/Aerator} = \frac{(100\ \text{HP})\ (1{,}000\ \text{gal})}{(0.15\ \text{HP})\ (7.48\ \text{gal/ft}^3)\ (12\ \text{ft})} = 7{,}427\ \text{ft}^2$$

Length = Width = 7,427 ft^2 = 86 feet

Design each basin with two rows of five aerators at 86 foot centers.

Length of Aeration Basin = 5(86 ft) = 430 feet

Width of Aeration Basin = 2(86 ft) = 172 feet

Use 175 foot width

Aeration Requirements (Oxygen Basin):

Calculate oxygen required based on a utilization rate of 2,164 lbs O_2/106 gal:

$$\text{Oxygen Required/Basin} = \frac{(2{,}164\ \text{lbs}\ O_2/10^6\ \text{gal})\ (72\ \text{MGD})}{(6\ \text{basins})}$$
$$= 25{,}968\ \text{lbs/day}$$

Calculate power requirements at a transfer efficiency of 2.9 lbs O_2/HP-hr:

$$\text{Power Requirements/Basin} = \frac{25{,}968\ \text{lbs/day}}{(2.9\ \text{lbs}\ O_2/\text{HP-hr})(24\ \text{hr/day})}$$

$$= 428\ \text{HP}$$

Aeration Requirements (Power Level Basis):

Calculate power requirements based on a minimum power level of 0.15 HP/1,000 gal:

$$\text{Power Requirements/Basin} = \frac{(36 \times 10^6\ \text{gal})(0.15\ \text{HP})}{(6\ \text{basins})\ (1{,}000\ \text{gal})}$$

Power Requirements/Basin = 900 HP

Since 900 HP is greater than 428 HP, power level controls; use ten 100 HP aerators and size basin according to power level.

Final Clarifier:

Basin Size

Calculate separate clarifications systems for each aeration basin with a maximum overflow rate of 700 gpd/ft^2:

Clarification Surface Area/Aeration Basin =

$$\frac{(72 \times 10^6 \text{ gpd})}{(6 \text{ basins})(700 \text{ gpd/ft}^2)} = 17,150 \text{ ft}^2$$

Calculate surface area using two clarifiers per aeration basin:

$$\text{Surface Area/Clarifier} = \frac{17,150 \text{ ft}^2}{2} = 8,575 \text{ ft}^2$$

$$\text{Diameter of Each Clarifier} = \frac{(4)(8,575)}{3.14}$$

Diameter of Each Clarifier = 109 feet

Use two 110 foot diameter clarifiers with a SWD of 10 feet, and check detention time minimum requirement of two hours:

Detention Time/Clarifier =

$$\frac{(3.14)(110 \text{ ft})^2 (7.48 \text{ gal/ft}^3)(10 \text{ ft})(24 \text{ hr/day})}{(4)(6 \times 10^6 \text{ GPD/clarifier})}$$

Detention Time = 2.84 hours, therefore adequate.

Sludge Return Pump Stations:

Design three communal pump stations, each serving four clarifiers with an operating recycle rate of 50 percent and a maximum recycle rate of 75 percent:

$$\text{Sludge Return Rate/Clarifier} = \frac{(0.50)(6.0 \times 10^6 \text{ gpd})}{(1{,}440 \text{ min/day})}$$

Sludge Return Rate/Clarifier = 2,080 gpm

Use three 1,000 gpm pumps per clarifier; two operational and one stand-by.

Design Summary

Aeration Basins:

Number of Basins = 6

Dimensions of Each Basin = 12 ft x 175 ft x 430 ft

Power Requirements = 60 - 100 HP aerators (10 each basin)

Final Clarifier:

Number of Basins = 12

Dimensions of each Basin = 10 ft SWD x 110 ft diameter

Sludge Return System:

Number of Pump Stations = three (each serving four clarifiers)

Pump Requirements = 36 - 1,000 gpm pumps (three per clarifier)

Cost Estimates (Secondary Treatment Facility)

Item		Est. Cost	
Concrete & Earthwork		$8,242,000	
Structural		679,000	
Mechanical		2,144,000	
Electrical		210,000	
Instrumentation & Controls		255,000	
Valves & Piping		382,000	
Contingencies & Miscellaneous		1,608,000	
	Total Capital		$13,520,000
Operating Costs		888,000	
Fixed Annual Cost		969,400	
	Total Annual		$ 1,857,400

Design of Effluent Polishing System (Fixed Bed Carbon Columns)

The conceptual design for an effluent polishing system using packed bed, pressure vessel carbon columns and the basis for design are described herein. The criteria as listed below are based on bench and pilot scale studies which are presented in Section VI of this report. The water quality of the columnar influent represents observed values of the pilot plant biological effluent. The quality numbers listed below represent higher, and thus more conservative, levels within the range of observed values.

Quality Criteria (Influent to Columns)

RANGE	DESIGN VALUE
COD (mg/l) 60 - 350	250
BOD_5 (mg/l) 20 - 120	100
Temperature (°C) 5 - 30	-
TSS (mg/l) 15 - 150	40
Oil Content, mg/l	<10

Process Requirements

(From bench and pilot studies and manufacturers' recommendations)

Flow = 72 MGD (50,000 gpm)
Linear Flow Velocity = 8 gpm/ft^2 (Section VI)
Contact Time (empty volume) = 20 minutes (Section VI)
Carbon Capacity = 0.40 lbs COD removed/lb carbon
(assume breakthrough = 80 mg/l COD)

Backwash Rate (no pre-filtration) = 15 gpm/ft^2

Required Carbon/Water Ratio for conveyance of spent and regenerated carbon = one lb carbon/gal of water
(per manufacturer's recommendation)

Carbon loss/regeneration cycle = 5% (per manufacturer's recommendation)

Reduction in original carbon capacity for 20 cycle operation = 10%
(per manufacturer's recommendation)

Regeneration Steam Equipment = one lb steam/lb carbon regenerated
(per manufacturer's recommendation)

Design Calculations

Carbon Columns:

Required Surface Area = $(50{,}000 \text{ gpm}) \left(\frac{\text{min} \times \text{ft}^2}{8 \text{ gal}} \right) = 6{,}250 \text{ ft}^2$

Use Standard 20 foot diameter column, Area = 314 ft^2

Required No. of Columns = $(6{,}250 \text{ ft}^2) \left(\frac{\text{column}}{314 \text{ ft}^2} \right) = 20$

Use parallel columnar operation, 20 sets. (2 columns per set)

Required empty bed carbon volume per set =

$$\frac{(2500 \text{ gpm/set})(20 \text{ min})}{(7.48 \text{ gal/ft}^3)} = 6{,}685 \text{ ft}^3$$

Required minimum carbon length per set = $\left(\frac{6{,}685 \text{ ft}^3}{314 \text{ ft}^2} \right) = 21.3 \text{ ft}$

Allow 50% expansion during backwash = 32 feet

Allow minimum of 7 feet per column for installation of inlet, backwash, and filter bottom appurtenances. Extra carbon depth allowances are made to allow single columnar operation while second column of series is being regenerated.

Use a series of two columns per set, 20 feet diameter x 25 feet deep.

Initial Carbon Inventory:

Initial Inventory/Set = $(6{,}685 \text{ ft}^3)(26 \text{ lbs/ft}^3) = 173{,}810$ lbs
Total Initial Inventory = 20 (173,810) = 3,476,300 lbs
Allowance for Idle Carbon Inventory = 400,000 lbs
Total Inventory = 3,876,000 lbs

Regeneration Requirements:

Virgin Carbon Capacity = 0.40 lbs COD removed/lb carbon
Average regenerated carbon capacity = 0.40 (.90) = 0.36
Design Loading (COD) = $(250 \text{ mg/l})(72 \text{ MGD})(8.34/10^6)$
= 150,000 lbs COD/day

COD Exhaustion Rate (assuming breakthrough COD = 80 mg/l)

$$\left(\frac{250-80}{250}\right)(150{,}000 \text{ lbs/day}) = 102{,}000 \text{ lbs COD/day}$$

$$\text{Regeneration Requirement} = \left(\frac{102{,}000}{0.36}\right) = 283{,}300 \text{ lbs carbon/day}$$

$$= 11{,}800 \text{ lbs carbon/hour}$$

Design regeneration furnace for this capacity. The final furnace selection will depend on carbon storage volume, furnace operating time, and feed rate as per manufacturer's recommendations.

Furnace Requirements:

Assume 90 lbs/day carbon to be regenerated per ft^2 hearth area:
(Largest furnace available 25' diameter x 12 hearth)

$$\text{Hearth Area} = \frac{(283{,}300 \text{ lbs carbon/day})}{(90 \text{ lbs carbon/ft}^2\text{/day})} = 3{,}100 \text{ ft}^2$$

$$\text{Steam Requirement} = (1 \text{ lb steam/lb carbon})\,(11{,}800 \text{ lbs carbon/hr})$$
$$= 11{,}800 \text{ lbs/hr}$$

Cost Estimate

Item	Est. Cost	
Earthwork and Concrete	420,000	
Inlet Lift Station	280,000	
Carbon Adsorber Tanks	5,040,000	
Slurry and Fresh Carbon Tanks	210,000	
Mechanical (pumps, comp., conveyance, screening)	280,000	
Piping and Valves	2,380,000	
Electrical	462,000	
Instrumentation and Control	336,000	
Structures	273,000	
Regeneration Furnaces (2) and Steam Generator	686,000	
Carbon Inventory	1,680,000	
Contingencies and Miscellaneous	1,518,000	
Total Capital		$13,565,000
Operating Costs	1,060,000	
Fixed Annual Cost	972,500	
Total Annual Cost		$2,032,500

Sludge Digestion and Dewatering

The selection of sludge handling processor was necessarily based on the ultimate disposal of the primary and wasted activated sludges. Since sludge disposal at sea is being curtailed and sludge incineration is not applicable with respect to the primary sludge, ultimate disposal by land fill was selected. (Reference Interim Pilot Plant Report, Chapter VII). Filter press dewatering was selected as the most applicable dewatering process since it will yield a sludge cake of sufficient dryness for direct landfill as opposed to alternate canidate process such as vacuum filtration and centrifugation (Reference Section VI of this Report). As a pretreatment step, gravity thickening of the primary and disgested waste activated sludge will be included. The wasted activated will be aerobically digested prior to dewatering.

Aerobic Digestion - Wasted Activated Sludge

Process Requirements:

Detention Time = 15 days (Section V) (Section VI)
Volume of Sludge = 500 lbs/10^6 gal
Reduction of Volatile Matter = 50 percent (Section VI)
Power Level in Basin = 0.15 HP/1000 gal

Design Calculations

Calculate Volume of Sludge
Volume = (500 lbs/10^6 gal) (72 MGD) = 36,000 lbs/day
Calculate Flow Based on One Percent Concentration

$$\text{Flow} = \frac{36{,}000 \text{ lbs/day} \times 10^6}{(8.34 \text{ lbs/gal})\ (10{,}000 \text{ mg/l})} = 432{,}000 \text{ gal/day}$$

Calculate Basin Volume with 15 Day Detention Time
Volume of Basin = (15 days) (432,000 gal/day)
= 6,470,000 gals

Using the basin dimensions of the activated sludge aeration basins.

Length = 430 ft.
Width = 175 ft.
Depth = 12 ft.
Volume = 6,750,000 gal

Use 10-100 HP floating type high speed aerators such that the basin depth can be varied.

Gravity Sludge Thickener

Process Requirements:

Loading Rate (not to exceed) = 10 lbs solids/ft^2 day
Primary Sludge Produced = 144,000 lbs/day
Digested Secondary Sludge Produced = 21,600 lbs/day (40% TSS reduction)

Design Calculations:

Calculate surface area of thickener using a loading of 10lb sludge/ft^2 day with a SWD = 10 ft.

$$\text{Surface Area} = \frac{\text{(165,600 lbs sludge/day)}}{\text{(10 lbs/ft}^2\text{/day)}} = 16{,}560\ \text{ft}^2$$

Use two 100 ft dia basins.

Calculate volume of sludge holding tank assuming a thickened sludge concentration of 4 percent.

$$\text{Volume} = \frac{\text{(162,000 lbs sludge/day) (10}^6\text{ gal)}}{\text{(8.34 lb/gal) (40,000 mg/l)}} = 486{,}000\ \text{gal}$$

Assuming 24 hr maximum detention time use one 100 ft dia sludge storage tank with mixer.

Filter Press

Process Requirements

Dry Solids Concentration of Cake = 45% (See Section VI)
Cake Density = 85 lb/ft^2
Total Sludge per Day = 162,000 lbs/day
Lime Dosage Required = 10% dry wt of sludge
Ferric Chloride Dosage Required = 5% dry wt of sludge

Design Calculations

$$\text{Cake Volume} = \frac{\text{(162,000 lbs) + (10) (162,000) + (0.05) (162,000)}}{\text{(85 lb/ft3) (0.45)}}$$

Cake Volume = 4,870 ft^3/day

Calculate volume of sludge per operating cycle assuming an effective operating time of 20 hrs/day - two hours per cycle.

$$\text{Cake Volume/Cycle} = \frac{4,870\ \text{ft}^3/\text{day}}{10\ \text{cycles/day}} = 487\ \text{ft}^3/\text{cycle}$$

Calculate plate requirements assuming 64 inches diameter press with a capacity of 2.4 ft^3 per plate.

$$\text{Number of Plates} = \frac{487\ \text{ft}^3/\text{cycle}}{2.4\ \text{ft}^3/\text{plate}} = 202$$

Select two 100 plate presses 64 inches in diameter.

Cost Estimates (Solids Handling)

Item	Est. Cost	
Earthwork	$141,000	
Concrete	883,000	
Piping and Valves	12,000	
Mechanical	268,000	
Structures	152,000	
Filter Press and Auxiliary Equipment	1,110,000	
Installation	186,000	
Lime Addition System	72,000	
Electrical	10,000	
Instrumentation and Control	82,000	
Contingencies and Miscellaneous	394,000	
Total Capital		$3,310,000
Operating Costs	$680,000	
Fixed Annual Cost	237,000	
		$917,000

SUMMARY

The design criteria, design calculations, and unit process cost estimates have been presented herein. A conceptual layout of the proposed treatment facility is shown in Figure 163. The summarized unit costs are tabulated in Table 52.

Figure 163

CONCEPTUAL LAYOUT OF THE DEEPWATER REGIONAL TREATMENT FACILITIES

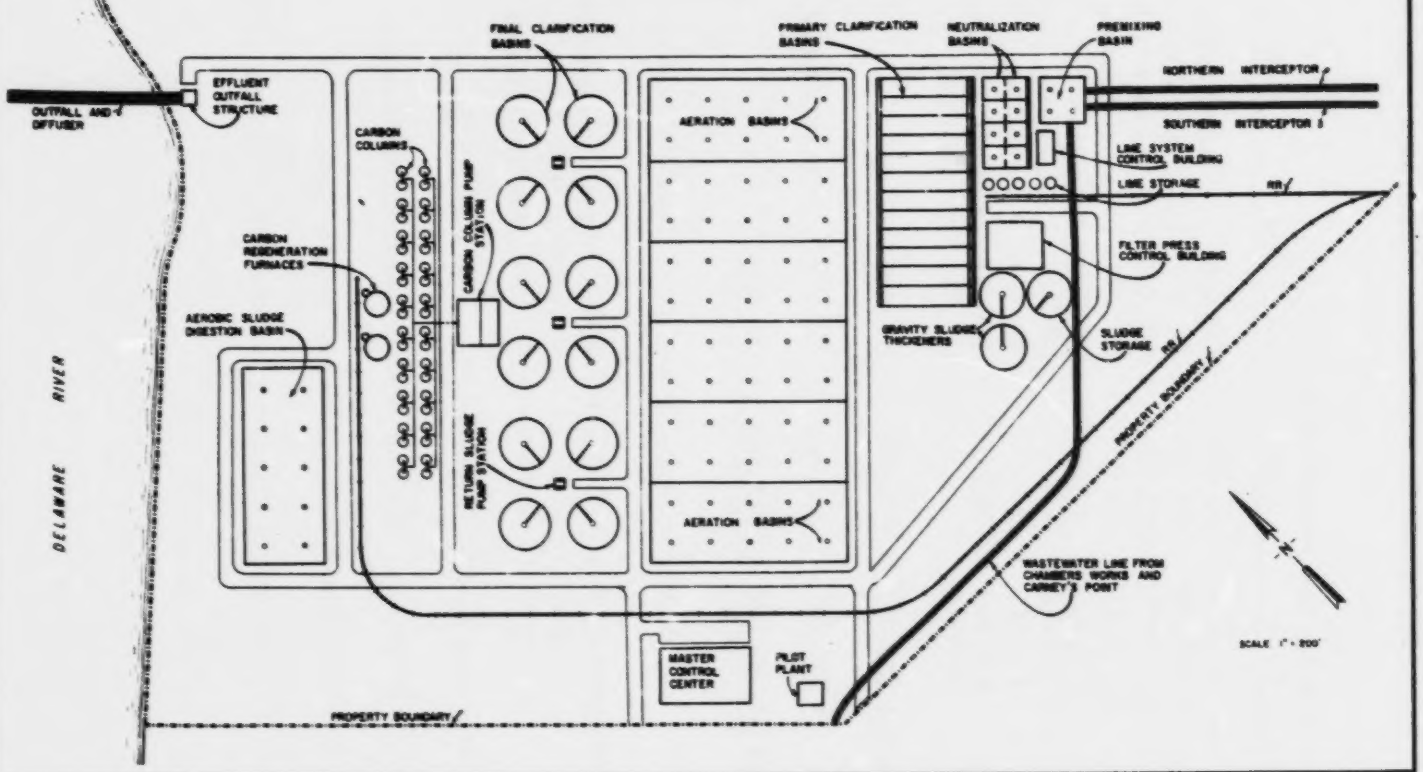

FINAL CLARIFICATION BASINS
PRIMARY CLARIFICATION BASINS
NEUTRALIZATION BASINS
PREMIXING BASIN
NORTHERN INTERCEPTOR
SOUTHERN INTERCEPTOR
LIME SYSTEM CONTROL BUILDING
LIME STORAGE
RR
FILTER PRESS CONTROL BUILDING
GRAVITY SLUDGE THICKENERS
SLUDGE STORAGE
RR
PROPERTY BOUNDARY
EFFLUENT OUTFALL STRUCTURE
OUTFALL AND DIFFUSER
CARBON COLUMNS
CARBON COLUMN PUMP STATION
CARBON REGENERATION FURNACES
AEROBIC SLUDGE DIGESTION BASIN
AERATION BASINS
AERATION BASINS
RETURN SLUDGE PUMP STATION
DELAWARE RIVER
N
WASTEWATER LINE FROM CHAMBERS WORKS AND CARNEY'S POINT
SCALE 1" = 200'
MASTER CONTROL CENTER
PILOT PLANT
PROPERTY BOUNDARY

TABLE 52

COST ESTIMATES FOR THE REGIONAL TREATMENT FACILITY[1]

Item	Construction Costs	Annual O&M Costs	Total Annual Costs	Description
1	$ 810,000	$ 165,000	223,000	Neutralization[2]
2	3,090,000	120,000	341,500	Primary Clarification[2]
3	13,520,000	888,000	1,857,400	Secondary Biological System[2]
4	3,310,000	680,000	917,000	Solids Handling and Disposal[3]
5	912,000	40,000	105,400	Electrical and Site Piping[4]
6	2,500,000	-	179,200	Foundation Work[5]
7	450,000	-	32,400	Re-routing of Henby Creek
8	800,000	12,000	69,400	Outfall Structure
9	1,000,000	-	71,700	Land Costs
Sub Total	$26,392,000	$1,905,000	$3,797,000	
10	13,565,000	1,060,000	2,032,500	Carbon Adsorption Effluent Polishing
Total	$39,957,000	$2,965,000	$5,829,500	

---

[1] Costs based on ENR of 1400 and include construction, engineering, legal, administrative, profit and contingencies. This ENR value used to be consistent with estimates cited in the Preliminary Engineering Report.

[2] Costs include ancillary appurtenances up to process limits.

[3] Cost includes sludge handling system - connection and controls.

[4] Electrical and piping costs outside unit process limits.

[5] Additional cost only if extensive pile foundations required.

SECTION VIII

EFFLUENT QUALITY ANALYSIS

The logical outgrowth of the bench and pilot scale treatability studies, the resulting formation of treatment concept, and the preliminary design of this system is to predict the quality of the effluent and relate it to the DRBC effluent quality standards. The activated sludge process followed by effluent polishing using activated carbon is deemed to be the most applicable system based on current technology for treating the combined wastewaters to a quality level commensurate with the DRBC objectives. This is predicated on the extensive bench and pilot work conducted pursuant to this project and the accompanying chemical and bio-chemical analyses.

The results of these bench and pilot studies in terms of effluent quality anlayses from the secondary activated sludge and the carbon column effluent polishing process are tabulated and summarized herein. They are then discussed interpretively with respect to the effluent quality standards as adopted by the Delaware River Basin Commission on March 7, 1968 and as amended through March 26, 1970. The interpretive guidelines adopted by the Commission on January 26, 1972 are shown in Table 53.

It is recognized that the effluent quality projection presented in this Section is based on the treatability of the combined wastewaters having the quality characteristics presented in this Report. However, the period of time over which the treated and untreated wastewaters were characterized affords statistical creditability. The effluent quality as predicted in this section is therefore sufficiently accurate to justify implementation of the recommended system which has the capacity to treat wastewaters of a similar nature to this quality level.

EFFLEUNT STANDARDS FOR THE REGIONAL TREATMENT FACILITY

The effluent criteria recently established by the DRBC are presented in Table 53.

DISCUSSION OF EFFLUENT QUALITY

The effluent quality as predicted from the bench scale tests is tabulated in Table 54. A more comprehensive quality analysis observed during summer and

TABLE 53

EFFLUENT QUALITY REQUIREMENTS
DELAWARE RIVER BASIN COMMISSION

Adopted January 26, 1972

1. Suspended Solids:

   For municipal and industrial waste treatment facilities, at least 90 percent removal as determined by an average of samples taken over each period of 30 consecutive days of the year and not to exceed 100 mg/l, whichever is less.

2. Public Safety:

   A. Temperature - Maximum 110°F where readily accessible to human contact.

3. Limits:

   A. Oil - not to exceed 10 mg/l; no readily visible oil.
   B. Debris, scum, or other floating materials - none.
   C. Toxicity -
      1) Not more than 50 percent mortality in 96 hours in an appropriate bioassay test with a 1:1 dilution. Wastes containing chlorine may be dechlorinated prior to the bioassay test.

      2) Notwithstanding the results of the tests prescribed in the stream quality objectives, the substances listed below being accumulative or conservative, shall not exceed the following specified limits in an effluent:

	Limit mg/l
Arsenic	0.1
Barium	2.0
Cadimum	0.02
Chromium (hexavalent)	0.10
Copper	0.20
Lead	0.10
Mercury	0.01
Selenium	0.02
Zinc	0.60

      3) Persistent pesticides - not to exceed one one-hundredth of the TL_{50} value at 96 hours as determined by appropriate bioassay.

   D. Odor - not to exceed a threshold number of 250.

   E. BOD -
      1) The former INCODEL Standards which were saved from repeal by Resolution 67-7 remain applicable; that is, no discharge shall exceed a daily average of 50 mg/l in Zone 1 and 100 mg/l in Zone 2. A slight deviation may be permitted by the Commission when it results from reduced secondary treatment plant efficiency caused by wastewater temperatures below 59°F (15°C).

      2) In Zones 2, 3, 4, and 5, a waste shall receive not less than zone percent reduction in addition to meeting allocation requirements.

These guidelines will be administered in accordance with the procedures contained in the Commission's Basin Regulations-Water Quality adopted 3/7/68.

TABLE 54

PREDICTED EFFLUENT QUALITY OF BIOLOGICAL TREATMENT BASED ON BENCH SCALE TEST*

QUALITY PARAMETER	MEAN VALUES	EFFLUENT RANGE
BOD_5 (filtered)	13 mg/1	6-30 mg/1
COD (filtered)	90 mg/1	60-250 mg/1
TOC (filtered)	65 mg/1	30-90 mg/1
Phenols	-	.01-0.30 mg/1
MBAS	-	7**
TKN	-	12-25 mg/1
$NO_2 + NO_3$-N	-	30-55 mg/1
Color	-	not measured
Heavy Metals	-	not measured

* Represent effluent quality levels using conventional biological treatment -- organic loading 0.5 lbs BOD/day/lb MLSS. Influent includes all industrial and municipal participants, proportionate to flow (Wastewater 510).

** Based on one analysis.

winter biological operations of the pilot plant is shown in Table 55. These data are discussed on a parametric basis. Table 56 presents the effluent quality of the pilot carbon columns.

Suspended Solids

The effluent suspended solids from the biological pilot plant ranged from 30 to 90 mg/l with a mean value of 52 mg/l. The carbon column effluent varied from 10 to 25 mg/l with a mean of 15 mg/l.

Temperature

The temperature of the effluent from the biological system ranged from approximately 5°C (41°F) during winter operations to 32°C (89°F) during the summer. There will be no significant deviation from this range in a full scale plant, although the winter effluent temperature is expected to be slightly higher based on heat balance calculations. The temperature of the biologically treated effluent will not be altered significantly through the polishing carbon columns.

pH

The influent to the regional plant, as in the pilot plant studies, will be neutralized to a pH in the 7 to 8.5 range. This pH will drop slightly in the secondary biological plant to a range of 6.5 to 8.0. No significant change in pH was observed through both the bench and pilot scale carbon columns, and this is expected to hold true for the full scale facility.

Oil

No oils of any consequence were noted in the composited raw wastewater throughout the treating program. Even if oils get into the interceptor, the activated sludge system can reduce oily substances from $\approx$ 50 mg/l to less than 10 mg/l. This system, coupled with carbon adsorption, should produce an effluent free of visible oil and less than 5 mg/l total oil.

Debris, Scum, Or Other Floating Materials

This criteria as established by the DRBC can be easily met by the proposed treatment system.

Toxicity

Toxicity tests were completed on the biological and activated carbon column effluents during the March, 1971 testing period. The toxicity tests were run

TABLE 55

OBSERVED EFFLUENT QUALITY OF THE PILOT BIOLOGICAL TREATMENT PLANT

	Summer Conditions		Winter Conditions	
PARAMETER	Mean	Range	Mean	Range
BOD_5 (filtered) mg/l	11	7-20	60	40-83
BOD_5 (unfiltered) mg/l	13	10-23	78	49-122
COD (filtered) mg/l	113	66-160	248	199-298
COD (unfiltered) mg/l	169	78-230	324	234-527
TOC (filtered) mg/l	39	22-57	77	60-93
TOC (unfiltered) mg/l	43	23-60	84	61-150
TOD (filtered) mg/l	113	50-172	233	164-292
TOD (unfiltered) mg/l	116	45-165	251	176-314

	Summer & Winter Conditions	
	Mean	Range
Kjeldhal Nitrogen, mg/l	24.2	9.5-47.0
Ammonia Nitrogen, mg/l	21.4	8.8-38.0
$NO_2 + NO_3$-N, mg/l	15.4	1.2-58.0
Total P, mg/l	0.95	0.1-3.9
Phenols, mg/l	0.75	0.04-8.00
Color, Standard Units	746	300-1,440
TSS, mg/l	52	30-90
TDS, mg/l	1,910	1,780-2,110
Sulfates, mg/l	510	448-575
MBAS, mg/l	3.2	2.2-4.2
Fecal Coliforms	0	-
Aluminum, mg/l*	0.44	0.3-0.7
Arsenic, mg/l	< 0.01	-
Cadmium, mg/l	< 0.02	< 0.01-0.03
Chromium (total) mg/l*	< 0.1	-
Chloride, mg/l	548	450-620
Copper, mg/l*	< 0.1	< 0.1-0.2
Fluoride, mg/l	0.248	0.04-0.54
Iron, mg/l*	< 0.32	< 0.1-1.1
Lead, mg/l*	< 0.1	< 0.1-0.2
Manganese, mg/l*	0.65	0.2-1.2
Mercury, mg/l*	0.00114	0-0.0050
Nickel, mg/l*	< 0.1	< 0.1-0.2
Silver, mg/l*	< 0.1	-
Strontium, mg/l*	0.41	0.3-0.6
Zinc, mg/l*	< 0.63	< 0.1-1.4

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* Sensitivity Limit of Analysis = 0.1 mg/l

TABLE 56

OBSERVED EFFLUENT QUALITY OF THE PILOT CARBON COLUMNS*

PARAMETER	MEAN	RANGE
BOD_5 (filtered) mg/l	20	10-36
BOD_5 (unfiltered) mg/l	25	17-40
COD (filtered) mg/l	62	29-102
COD (unfiltered) mg/l	94	33-204
TSS, mg/l	15	10-25
Color, Standard Units	100**	0-100
Phenols, mg/l	0.09	0.05-0.15
MBAS, mg/l	0.15	0.05-0.20

* Data generated during winter operations

** Color breakthrough occurred after COD breakthrough, therefore color during column operation would be <100 color units.

in accordance with the procedures described in the FISH-PESTICIDE ACUTE TOXICITY TEST METHOD prepared by the Environmental Protection Agency and the Fish Bioassay Procedure described in the 1970 edition of Standard Methods (APHA).

The toxicity tests were made utilizing fathead minnows (Pimephales promelas) acquired from a commercial hatchery in Arkansas and had a mean weight and length of 0.96 oz. and 37 mm, respectively.

The test fish were observed in the laboratory hatchery facilities for at least 10 days prior to testing. During that period, mortality in the test populations was less than 2 percent and the fish were judged to be in excellent physical condition. Bioassays were conducted in five gallon glass vessels held in constant temperature (18°C ± 0.5) water baths. The test diluent consisted of 15 liters of deionized water of at least one million ohms resistivity which was reconstituted by adding three mg potassium chloride, 30 mg calcium sulfate, 30 mg magnesium sulfate, and 48 mg sodium bicarbonate per liter. The pH of the diluent was 7.1, and the methyl orange alkalinity was 35 ppm. Bioassays were conducted under static conditions, without aeration, and with a single introduction of the effluent in question. Fish of any one species were of approximately the same weight and length (± 20%). Fish were conditioned to the test water for at least 24 hours prior to testing. Test solutions were prepared by adding appropriate amounts of effluent to sufficient test diluent to yield a final test volume of 15 liters. The dissolved oxygen levels in the effluent tested was never less than 5.2 mg/l. The test diluent was saturated prior to use in a bioassay by bubbling oxygen through it. Ten fish were tested at each concentration, the mass/volume ratio never exceeded 1.0 gram of fish per liter of water. A minimum of seven concentrations of the chemical formulation were prepared in logorithmic series and used to evaluate the susceptibility of each fish species to each compound.

The 96 hour TL_{50} values (95 percent confidence interval) were obtained on the six hour aeration effluent, the 12 hour aeration effluent and the 12 hour aeration effluent treated with granular activated carbon. The six hour aeration effluent TL_{50} values at 96 hours averaged 15.9 percent wastewater in the test solution. The 12 hour aeration effluent TL_{50} values at 96 hours averaged 30.0 percent wastewater in the test solution. The activated carbon effluents showed no toxic effects at 96 hours and therefore, since all the fish were alive after 96 hours, no TL_{50} values were obtained. Hence the effluent from the carbon columns will meet or exceed the effluent quality as set forth in Table 54.

Odor

Odor tests were completed on the biological and carbon effluents during the

March, 1971 testing period. The threshold odor numbers for the biological effluent ranged from 200 to 800, with a geometric mean of 346. In comparison, the carbon effluent threshold odor number was four based on a 24 hour composite sample.

BOD

Extensive effluent BOD_5 information is available from these bench and pilot scale treatability studies. As noted in Tables 54 and 55, the BOD_5 of the biologically treated effluent can be expected to range from 7 to 30 mg/l during summer operations and as high as 122 mg/l during the most severe winter conditions. If carbon adsorption is used as an effluent polishing step, this can be reduced to a BOD_5 concentration of less than approximately 25 mg/l throughout the operating year.

During the course of this investigation, a series of BOD analyses were run in order to tabulate biochemical oxygen demand versus time. The objectives of obtaining this information were (1) to determine first-stage biochemical oxygen demand reaction rate K, (2) to determine first stage ultimate oxygen demands, and (3) to use the information thus obtained to predict first stage ultimate oxygen demands (FSOD).

In order to accomplish the above objectives, BOD data obtained during the months of February and March, 1971, were analyzed by several techniques. Both the rapid ratio method and the method of moments were used to ascertain first stage BOD reaction rates and first stage ultimate oxygen demands. Additionally, k rates developed from this winter operations data were compared for similarity with data obtained during previous summer operations.

The BOD data used for this study are tabulated in Table 56. It should be noted that BOD's were taken at intervals of 1, 3, 5,7, 11, 15 and 20 days, thus allowing a BOD vs time relationship to be developed. All samples used were inhibited against nitrification. Therefore, the first stage biochemical oxygen demand being measured should have approached the first order reaction mathematically described by Equation VIII-1.

$$y = L(1-10^{-kt}) \qquad \text{VIII-1}$$

where:

y = biochemical oxygen demand exerted at time t
L = first stage ultimate oxygen demand
k = reaction rate constant
t = time

Analyzing the data using the two techniques mentioned in the preceding, the following results were obtained:

1. Using the rapid ratio method --
   k_1 (avg) = 0.059 day^{-1}, L = 214 mg/l

2. Using the method of moments -
   k_1 (avg) = 0.080 day^{-1} , L = 214 mg/l

3. Using the average of the above two determined values -
   k_1 (avg) = 0.070 day^{-1}, L = 214 mg/l

The average rate constant as determined above compared favorably with rate constants developed from BOD data obtained during operations during the summer of 1970. It was therefore possible to use the rate constants developed to determine FSOD (first stage oxygen demands) for winter and summer operations assuming an average BOD_5 of 20 mg/l. This was done by means of the following calculation and yielded an average FSOD of 36.2 mg/l.

since $y = L(1-10^{-kt})$
$20 = L(1-10^{-5k})$

and

$$L = \frac{20}{.553} = 36.2 \text{ mg/l}$$

Using the average k_1 value of 0.070 day^{-1}.

This therefore indicates that during summer operations if effluent five-day BOD's are maintained at 20 mg/l, the FSOD should not exceed 36 mg/l on the average. This predicted value compares quite favorably with measured BOD_{20} values of from 25 to 30 mg/l during summer operations. This is a conservative approach in that k values tend to decrease with an increasing degree of biological treatment. This means that when the effluent is of better BOD quality than that reported in Table 55, $FSOD/BOD_5$ ratio will tend to decrease toward unity.

The FSOD for winter operations can be calculated in the following manner. The k value of 0.070 day^{-1} is referenced to the standard incubation temperature of 20°C and therefore can be used to correct five day BOD value to FSOD levels at any temperature. Assuming the BOD_5 of the biological treated effluent during winter operations ranges between 49 and 122 mg/l, an FSOD range between 90 and 220 mg/l could be expected.

As previously stated, the biological system followed by a polishing step using carbon columns is capable of producing an effluent having a BOD_5 of less than 15 mg/l during summer operations. This level is not expected to materially increase during the winter months because carbon capacity is available to handle the increased organic loading to the columns. On this basis, the final effluent FSOD can be expected to range between 10 to 35 mg/l throughout the year.

Color

Color levels of the biological and carbon effluents were measured on the platinum-cobalt scale. The effluent color from the biological system is included in Table 55. The mean value of 746 units exceeds regulatory criteria. Excellent color removal was effected, however, in the bench and pilot scale carbon columns as indicated in Section VI. Based on these data, the color of the carbon column effluent will be below 100 standard units on the platinum-cobalt scale.

Trace Organics

Phenols were monitored through both the biological system and the carbon columns. Phenols and organic compounds exhibiting "phenolic" characteristics are both biodegradable and sorbable. This is confirmed by the phenol carbon isotherms shown in Section V, the bench scale carbon studies shown in Section VI, Figure 127, and the biological removal indicated in Table 55. Based on this data, the biologically treated effluent will have a phenol concentration in the 0.04 to 8.00 mg/l range and the phenols will be less than .05 mg/l in the carbon column effluent.

Many of the miscellaneous trace organics will be removed to levels below detection limits in the carbon columns, with the exception of refractory compounds. There is nothing to indicate, however, that these refractions will cause any deleterious effect on the water body receiving the treated effluent.

Inorganic Constituents

Little change in the level of dissolved inorganic constituents through the biological-carbon system can be anticipated. Based on the composite wastewaters used in this study, the effluent from the biological and carbon units will contain a TDS of 1700 to 2200 mg/l, fluorides of <1.0 mg/l, chlorides of 400 to 650 mg/l, sulfates of 400 to 600 mg/l, and nutrites-nitrates of 1 to 60 mg/l. These concentrations in the biologically treated effluent are reported in Table 55.

Nutrients

The nitrogen and phosphorus levels in the biologically treated effluent are shown in Table 27. These are reported in terms of TKN (ammonia and organic N) with a mean value of 24 mg/l; ammonia-nitrogen, which has a mean level of 21 mg/l; and total phosphorus, which has a mean level of 1.0 mg/l. Based on the ammonia analyses through the biological plant, little nitrification occurred within the 12 hour detention time. This indicates that biological effluent ammonia will be highly dependent on the influent concentration. Moreover, no significant degree of ammonia removal can be expected through the carbon columns as indicated in Section VI. The phosphate concentration will remain relatively unchanged through the carbon columns.

Fecal Coliforms

No fecal coliforms were observed in the effluent from the pilot plant at any time during the study as shown in Table 55.

Heavy Metals

Heavy metallic ions were analyzed using an atomic adsorption spectrophotometer through the pilot plant treatabilitity studies. The average values for 12 different metals in the biologically treated effluent are listed in Table 55. The levels indicated therein are commensurate with the accuracy of the analytical equipment used to perform those analyses. It is noted that the most sensitive analytical capability was for mercury, where levels as low as one part per billion could be detected.

Only a slight decrease in metallic ion concentration can be anticipated in the effluent polishing step based on observed data. This slight removal is most probably attributable to sorption of organic-metallic complexes, or organic compounds with metallic functional groups.

Radioactivity

The level of radioactive substances in the biological and carbon effluents were analyzed. Both gamma and gross beta radiation levels were determined. The gamma radiation activity (photons originating from nucleii of excited atoms) is indicated in Table 57. Gross beta levels (negation of nuclear origin) are presented in Table 58. The radioactivity indicated represent normal levels well below hazardous thresholds. It is interesting to note the removal of both beta and gamma emitting substances in the carbon column.

TABLE 57

GROSS GAMMA ANALYSES (0-2.56 MeV)

	Suspended				Dissolved			
Sample Description	A1	A2	B1	B2	A1	A2	B1	B2
Weight kg	-	-	-	-	.648	654	667	612
nCi/kg (K-40 equivalent)	neg	neg	0.15 (±11)	0.3 (±12)	2.7 (±1)	1.2 (±1)	neg	0.5 (±1)
nCi/kg (γ equivalent)	neg	neg	0.2 (±.01)	.01 (±.01)	0.2 (±1)	0.1 (±1)	neg	0.06 (±.1)

No spectral peaks were observed except for K-40 in A dissolved.

"A" Samples = Biologically treated effluent

"B" Samples = Carbon column effluent

TABLE 58

GROSS BETA ANALYSES

Gross β'

20 ml sample volumes evaporated on stainless steel planchets and counted on Nuclear Chicago Low Background proportional counter.

Sample #	Activity pCi/1	
A-1	28.4 ± 6.1	20.3
A-2	12.2 ± 5.8	
B-1	13.5 ± 5.8	15.8
B-2	18.0 ± 5.9	

Lower limits of detection: χ = .025 β = .050

MSMA = 7.65 pCi/1

MDTA = 14.07 pCi/1

"A" Samples = Biologically treated effluent

"B" Samples = Carbon column effluent

SUMMARY

The predicted effluent quality from both the biological treatment facility and the carbon columns have been discussed in this Section. These values, which correspond to the quality of the composited raw wastewater used in this testing program and the stated treatment conditions, are tabulated in Table 59.

TABLE 59

PREDICTED EFFLUENT QUALITY

Constituent	Primary Effluent	Activated Sludge Effluent	Combined Activated Sludge-Carbon Effluent	Remarks
BOD_5, mg/1	150-340	7-122	< 20	BOD residual depends on BOD/COD ratio which characterizes relative biodegradability of wastewater.
FSOD, mg/1	-	12-220	10-35	
COD, mg/1	400-800	80-230	< 80	Exact COD residuals vary with complexity of wastewater & design contact times in the Act. S. and Carbon Treatment Plants.
pH	7-8.5	6.5-8	6.5-8	pH drop in Act. S. systems attributed to biological production of CO_2 and intermediate acids. pH change in carbon columns depends on preferential adsorption of acidic and basic organics.
Temperature, °C	7-33	5-32	No significant removal	
TSS, mg/1	20-40	20-50	10-25	
TDS, mg/1	1,780-2,200	1,780-2,200	No significant removal	TDS is essentially unchanged through all three treatment systems.
Toxicity (Bioassays TL_{50} @ 96 hr) (% wastewater)	.	30%	100%	
Oil	10-20	< 10	< 5	
Color, standard units	300-1,440	300-1,440	< 100	
Odor	-	200-800	≈4.0	
MBAS, mg/1	2.2-4.2	2.2-4.2	< .40	
Phenols, mg/1	0.5-15	0.05-10	< 0.05	Phenols(ics) are generally amenable to biological and sorption removal.
Chloride, mg/1	400-650	400-650	No significant removal	
Fluoride, mg/1	< 1	< 1	< 1	
NH_3-N, mg/1	8.8-38	8.8-38	No significant removal	
TKN-N, mg/1	9.5-47	9.5-47	No significant removal	
Total P, mg/1	0.1-3.9	0.1-3.9	No significant removal	

TABLE 59 cont'd.

PREDICTED EFFLUENT QUALITY

Constituent	Primary Effluent	Activated Sludge Effluent	Combined Activated Sludge Carbon Effluent	Remarks
Fecal Coliforms	0	0	0	
Radioactive Substances				
Gamma nCi/kg		0.1-0.2	0-0.6	
Beta pCi/l		20.5	15.8	
Heavy Metals				Values reported are based on analysis as shown in Figure VI-2.
Aluminum, mg/l	0.3-0.7	No significant removal	Possibility of slight removal	
Arsenic, mg/l	< 0.01	No significant removal	Possibility of slight removal	
Cadmium, mg/l	< 0.02	No significant removal	Possibility of slight removal	
Chromium (Total), mg/l	< 0.1	No significant removal	Possibility of slight removal	
Copper, mg/l	< 0.1-0.2	No significant removal	Possibility of slight removal	
Iron, mg/l	< 0.1-1.1	No significant removal	Possibility of slight removal	
Lead, mg/l	< 0.1-0.2	No significant removal	Possibility of slight removal	
Manganese, mg/l	0.2-1.2	No significant removal	Possibility of slight removal	
Mercury, mg/l	0-0.0050	No significant removal	Possibility of slight removal	
Nickel, mg/l	< 0.1-0.2	No significant removal	Possibility of slight removal	
Silver, mg/l	< 0.1	No significant removal	Possibility of slight removal	
Strontium, mg/l	0.3-0.6	No significant removal	Possibility of slight removal	
Zinc, mg/l	< 0.1-1.4	No significant removal	Possibility of slight removal	

APPENDIX A

STATPK COMPUTER PROGRAM

ELECTRONIC DATA ANALYSIS AND PROCESSING

The large quantity of data accumulated during this extensive wastewater characterization and biological waste treatment investigation makes rapid and reliable data handling and analysis techniques indispensable. It is valuable for the user of these data to know the statistical reliability of his information. The development of the design parameters and coefficients for biological waste treatment processes involves numerous mathematical manipulations which are both time-consuming and subject to computational error. It is also informative to determine the error inherent in the design coefficients and parameters to reduce the uncertainty in process design calculations. Unfortunately, application of the theory of propagation of errors to field data is a time-consuming process and is thus usually neglected in biological waste treatment investigations.

The availability and utility of high-speed electronic computers gives the environmental engineer a tool which he can use to relieve himself of tedious and complicated mathematical procedures. In view of the myriad of data accumulated during the bench and pilot scale phases of this project, a computer program was developed to perform the necessary mathematical operations on biological waste treatment process information and to arrive at the required design information and the errors associated with it. This program provides the user with the following analyses:

1. Analysis of user-selected parameters to determine if steady-state conditions prevailed during the sampling period.
2. A statistical analysis of each parameter for the sampling period.
3. Removal of outliers from the original data (for each parameter) and a recomputation of the statistics.
4. Computation of biological waste treatment process parameters (organic loading, removal rates, etc.) and their associated most probable errors.
5. Least squares curve fitting of process parameters to obtain design coefficients (a, a', b, b'). Correlation coefficients are developed to indicate the error

in these coefficients.

6. Plotting graphically either with a pen plotter or line printer, the design parameters and coefficients previously computed.

The computer program was developed for use on a high-speed, large core computer such as the UNIVAC 1108, CDC 6400, or IBM 360/50. With the exception of the plotting routines, the program is machine independent. Most of the typical data analysis problems solved with this program should compile and execute in considerably under one minute on any of the above machines. This results in a considerable savings in manpower as well as permitting a better statistical analysis with a reduced opportunity for error.

The following paragraphs briefly consider each of the aforementioned analyses performed by Program STATPK which is schematically illustrated in Figure A-1. The reader is referred to the bibliography and user's manual if additional information on the computational algorithm is desired (References 1, 2, and 3).

Data Input

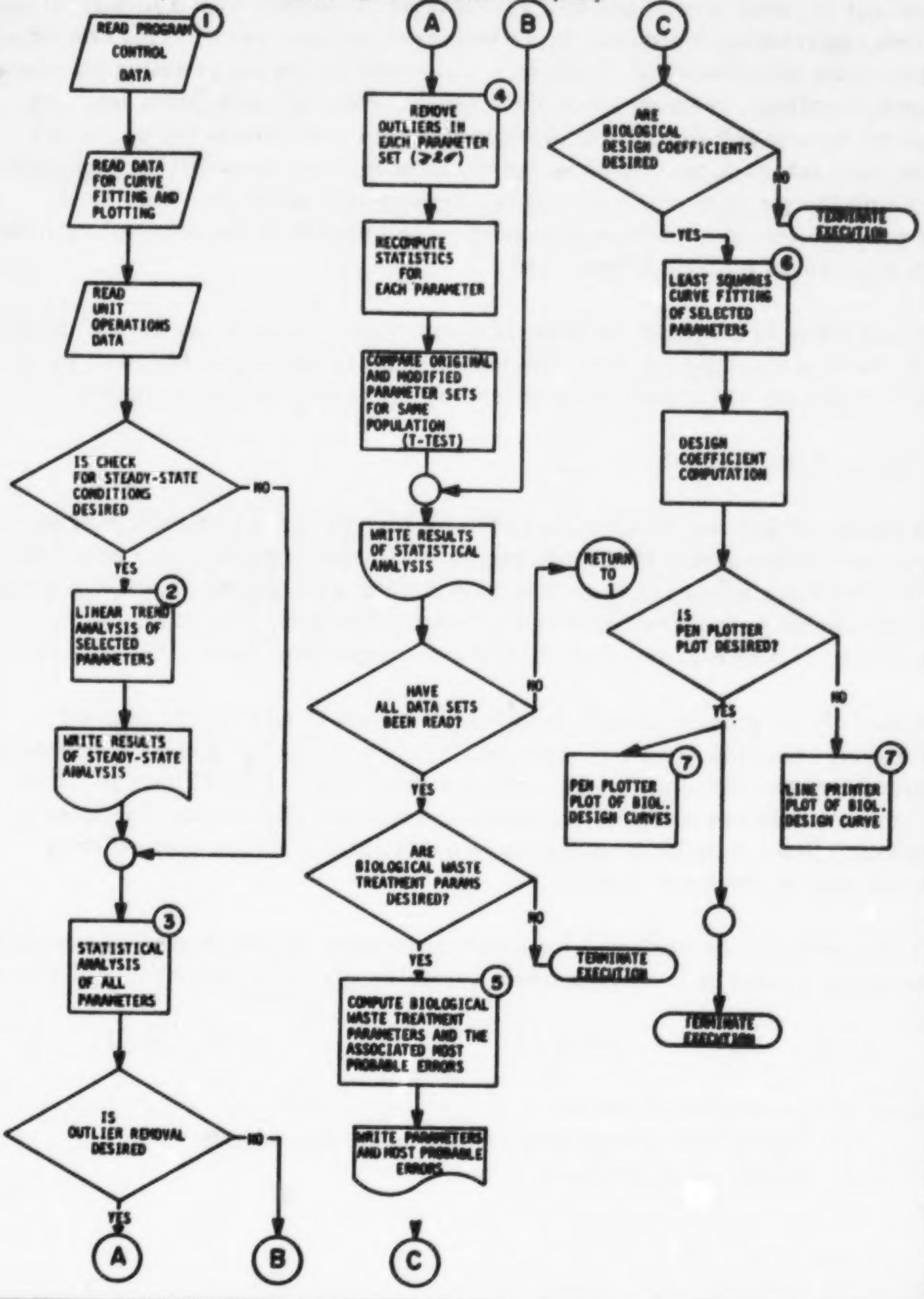
Program STATPK is user-oriented and is thus relatively simple for an individual to use with only a basic knowledge of computers and FORTRAN. The input data are written on specially designed coding forms to facilitate coding and keypunching. A maximum of 32 different input parameters, not including the date of sampling, are used in this program. These parameters are:

Influent*	Effluent*	Mixed Liquor
Total BOD_5	Total BOD_5	Waste Flow Rate (liters/day)
Total COD	Total COD	Aeration Volume (liters)
Total TOD	Total TOD	TSS*
Total TOC	Total TOC	VSS*
Soluble BOD_5	Settled BOD_5	Oxygen uptake (mg/l/hr)
Soluble COD	Settled COD	Temperature (°C)
Soluble TOD	Settled TOD	Waste Solids (gms/day)
Soluble TOC	Settled TOC	
TSS	Soluble BOD_5	
VSS	Soluble COD	
IOD	Soluble TOD	
	Soluble TOC	
	TSS	
	VSS	

* mg/l unless otherwise indicated

Figure A-1

SCHEMATIC OF COMPUTATIONAL TECHNIQUE PROGRAM STATPK



A
B
C
READ PROGRAM CONTROL DATA
①
READ DATA FOR CURVE FITTING AND PLOTTING
READ UNIT OPERATIONS DATA
IS CHECK FOR STEADY-STATE CONDITIONS DESIRED
NO
YES
LINEAR TREND ANALYSIS OF SELECTED PARAMETERS
②
WRITE RESULTS OF STEADY-STATE ANALYSIS
STATISTICAL ANALYSIS OF ALL PARAMETERS
③
IS OUTLIER REMOVAL DESIRED
NO
YES
A
B
REMOVE OUTLIERS IN EACH PARAMETER SET (≥2σ)
④
RECOMPUTE STATISTICS FOR EACH PARAMETER
COMPARE ORIGINAL AND MODIFIED PARAMETER SETS FOR SAME POPULATION (T-TEST)
WRITE RESULTS OF STATISTICAL ANALYSIS
RETURN TO 1
HAVE ALL DATA SETS BEEN READ?
NO
YES
ARE BIOLOGICAL WASTE TREATMENT PARAMS DESIRED?
NO
TERMINATE EXECUTION
YES
COMPUTE BIOLOGICAL WASTE TREATMENT PARAMETERS AND THE ASSOCIATED MOST PROBABLE ERRORS
⑤
WRITE PARAMETERS AND MOST PROBABLE ERRORS
C
ARE BIOLOGICAL DESIGN COEFFICIENTS DESIRED
NO
TERMINATE EXECUTION
YES
LEAST SQUARES CURVE FITTING OF SELECTED PARAMETERS
⑥
DESIGN COEFFICIENT COMPUTATION
IS PEN PLOTTER PLOT DESIRED?
YES
NO
PEN PLOTTER PLOT OF BIOL. DESIGN CURVES
⑦
LINE PRINTER PLOT OF BIOL. DESIGN CURVE
⑦
TERMINATE EXECUTION

Any or all of the above parameters are entered on a coding form for each sampling period. If a parameter is not sampled or is not to be included in the computations, a negative one is entered in its position on the appropriate coding form. This is necessary since the program would otherwise use a zero value in the statistical computations. Three coding forms, designated Files 1, 2, and 3, are filled out for each sample period. A "data set" is formed from a number of sample periods representing biological waste treatment process operation for one set of steady-state conditions. A "data set" will result in one set of design parameters (organic loadings, removal velocities, etc.). Three or more "data sets" are required for computation of the biological design coefficients (a, a', b, b') since each set produces one point for the least squares curve-fitting technique. Although the program would fit a curve through two points (two data sets), this practice should be considered undesirable because of the uncertainty inherent with the limited amount of data used.

The user reads in a number of these data sets, each separated by an end-of-file card, for a computational run. The last data set to be read is followed by an end-of-job card which indicates the end of the problem to the computer.

Steady-State Analysis

The theory behind the calculation of the biological waste treatment process design parameters and coefficients assumes that steady-state (with respect to time) conditions prevailed when the process data were taken. Since this assumption is fundamental to the development of these coefficients, it is advisable to determine, if possible, the existence of time-dependent trends in a data set.

The user of this program selects anywhere from one to four input (process) parameters which he feels would be most likely to show the presence or absence of steady-state conditions (e.g., effluent total COD, MLVSS, etc.). It is also desirable to use essentially equally-spaced data with as many samples as possible. These conditions assure maximum reliability of the curve-fitting process used in the trend analysis.

The parameters to be andlyzed for trends are treated as the dependent variable with, time being treated as the independent variable, giving an equation of the form:

$$y = a + bt \qquad \text{(A-1)}$$

where: y = parameter of interest
t = incremental sample time, $t = 0$ for the first sample
a, b = regression coefficients

Least squares regression is used to fit this simple linear function to the data. The coefficient "b" represents the slope, which defines the time-dependent trend. A positive slope indicates that the parameter value was increasing with time while a negative slope denotes the opposite condition. Obviously, this provides the engineer with a reasonable assessment of the stability of the process during the sampling period.

However, merely fitting a linear function to the time series data to discern trends does not provide the analyst with information pertaining to the reliability of the trend analysis. Thus it is necessary to incorporate a technique to evaluate the significance and reliability of the trend coefficient (slope). The method used in this program for this purpose involves the computation of a "t-value" for the regression coefficient which is a means for arriving at the confidence intervals of the coefficient. The equation for computing the t-value of the regression coefficient is:

$$t = (b - B) \; \frac{(n - 2) \quad \Sigma(x_i - \hat{\bar{x}})^2}{\Sigma(Y_i - Y_i')^2} \qquad \text{(A-2)}$$

where:

b = estimated least squares regression coefficient
B = true regression coefficient
x_i = independent variable
Y_i = sample dependent variable
Y_i' = population dependent variable
$\bar{x}$ = mean of variable x (estimated)

It can be shown that this t-value for the regression coefficient possesses a "Student's t" distribution with "n-2" degrees of freedom. The program essentially tests the hypothesis that:

$$H_0 : B = b$$

against the alternative:

$$H_1 : B \neq b$$

The t-value computed from the preceding equation is compared with a standard table of "Student's t" values for the appropriate confidence levels and degrees of freedom. For example, ten (10) samples at the 5 percent confidence limit would have a t-value of 2.36. In other words, to have 95 percent certainty that a trend coefficient was significant would require a t-value of 2.36 or greater for ten (10) samples.

The utility of this technique for steady state analysis is obvious. By specifying appropriate parameters for analysis, the engineer can rapidly and reliably detect any time dependent trends by this program and a table of "Student's t" distribution. The program user is referred to any standard statistics text for additional information on this analysis.

Statistical Analysis

Program STATPK performs a complete statistical analysis on all process parameters read as input. The statistical analyses used are based upon the theories of random sampling and small sample distributions, which are also applicable to large sample sizes. In order to simplify computational procedures the collected data are assumed to follow a normal Gaussian distribution. With the exception of parameters which may frequently have values close to or equal to zero, this assumption should be generally adequate for biological waste treatment data. The user should be aware that the normality assumption is liable to fail under certain conditions and should use reasonable care and judgment in the application of this data analysis package.

The statistical characteristics computed for each parameter are the mean:

$$\bar{x} = \frac{1}{n} \sum_{i=1}^{n} x_i \qquad \text{(A-3)}$$

where: $\bar{x}$ = mean of parameter x (estimated)
x_i = ith datum of parameter x
n = number of samples;

the standard deviation:

$$\sigma = \sqrt{\frac{(x_i - \bar{x})^2}{n - 1}} \qquad \text{(A-4)}$$

where: σ = standard deviation of parameter , x
the coefficient of variation:

$$CV = \frac{\sigma}{\bar{x}} \times 100 \qquad \text{(A-5)}$$

where: CV= coefficient of variation, in percent, and the standard deviation of the mean:

$$\sigma_{\bar{x}} = \frac{\sigma}{n} \qquad \text{(A-6)}$$

The four preceding statistical measurements provide the program user with a quantitative estimate of the validity of the process data. The arithmetic mean, $\bar{x}$, of a series of samples of a given parameter is the most probable value of that parameter. It can be shown that the arithmetic mean is the best unbiased estimate for the true mean of a normally distributed population. The mean also is generally superior to the mode and median as a measure of central tendency for other types of distributions because it usually tends to be more stable than these other measures of location.

The standard deviation, σ, is a measure of variation or dispersion in a sample population of a parameter. Standard deviation is a measure of the probability that a single reading will be near the sample mean value. For most common types of data, the standard deviation is superior to other common measures of variation due to its greater stability in repeated sampling experiments, which is similar to the situation of the mean with respect to other measures of location.

The coefficient of variation, CV, provides the analyst with a measure of a relationship of the variation in a sample population (σ) to the magnitude of the numbers observed ($\bar{x}$). This measure indicates whether an increase in variation (σ) is due to large magnitudes of the parameter being sampled or to some other influence, such as sampling error. It is also useful in comparing the variability of parameters which are measured in different units. However, the coefficient of variation is not a rigorous statistical measure and should not be used to attempt to quantify the sizes of variations between sample populations of parameters.

The final statistical measure computed by this program is the standard deviation of the mean, $\sigma_{\bar{x}}$. As was previously discussed, the standard deviation, σ, is a measure of the reliability of a single sample with respect to the mean of the sample population. The standard deviation of the mean is a measure of the reliability of the estimated mean, $\bar{x}$, as a predictor of the true population mean, μ. The basis for this statistic is that the mean of "n" equally precise observations is a much more reliable estimate of the population mean than any single observation. The standard deviation of the mean is also useful in estimating probable errors of products and quotients involving means of various parameters and data sets.

All of the above statistics are computed for each parameter in each data set. After the statistics are computed, each parameter sample population is searched for "outliers." Outliers are defined as sample values for which the probability of occurrence is so low that these values can be considered to be in error and can thus be discarded from the sample population. An arbitrary cutoff limit of 1.96 standard deviations is used in this program for outlier reduction. This value corresponds to the 95 percent confidence interval for a Gaussian distribution. By using 2.58 standard deviations for the cutoff criterion, the analyst could increase this certainty to 99 percent. The search for outliers in each parameter sample population is conducted using the mean and standard deviation computed for that population. A new set of statistics is then computed for the modified (outliers removed) sample population of each parameter. If a parameter set is found to have no outliers, this operation is bypassed.

It is mandatory that the modified sample population be representative of the "true" population of the parameter being considered if meaningful results are to be obtained in subsequent calculations. To ascertain whether or not the modified parameter data are still representative of the original sample population, a form of "Student's t" test is used to compare the means of the two data sets (original and modified). This test requires an assumption that the variances of the two populations are equal and tests the hypothesis:

$$H_o : \mu_x = \mu_y$$

against the alternate:

$$H_1 : \mu_x \neq \mu_y$$

where: μ_x and μ_y denote the "true" means of the original and modified sample populations, respectively. The t-value is then computed using the equation:

$$t = \frac{(\bar{x} - \bar{y}) - (\mu_x - \mu_y)}{\sqrt{n_x S_x^2 + n_y S_y^2}} \sqrt{\frac{n_x n_y (n_x + n_y - 2)}{n_x + n_y}} \qquad \text{(A-7)}$$

where:

$\bar{x}$ = estimated mean of original sample population
y = estimated mean of modified sample population
μ_x, μ_y = true parameter population means (unknown)
S_x = estimated standard deviation of original sample population
S_y = estimated standard deviation of modified sample population
n_x = no. of samples used to estimate x, S_x
n_y = no. of samples used to estimate $\bar{y}$, S_y

which can be shown to have "Student's t" distribution with "$n_x + n_y - 2$" degrees of freedom. The computed t-value is tested in a manner similar to that previously shown for the regression coefficient in the trend analysis. If the alternative hypothesis ($\mu_x \neq \mu_y$) is shown to be valid, the modified data set cannot be used for any additional calculations since it is not representative of the sample population. In this case, the original sample data are used in the computation of biological design parameters. If the modified data set passes the t-Test, it is used for these calculations rather than the original data. The removal of outliers is performed to attempt to eliminate bias in the data due to experimental or sampling error.

At this point in the data manipulations, the program now has a mean, standard deviation, coefficient of variation and standard deviation of the mean for each parameter in each data set. The statistics may be either from the intitial sample populations or the sample populations with the outliers removed. The data used are appropriately flagged to indicate: a) no outliers present and original data used, b) outliers present and modified data used, and c) outliers present but original data used due to significant changes in the modified data. These statistics are then used throughout the remainder of the program to represent each parameter in all subsequent computations.

Compute Biological Waste Treatment Design Parameters

The user of program STATPK has the option to calculate a number of biological waste treatment process parameters for use in process evaluation and design (Reference 4). These parameters are listed in Table A-1.All computations leading to these parameters are performed in accordance with established engineering practice. Each data set provides one (1) value for each of the design parameters and these values are combined with similar values from other data sets to compute the biological design coefficients, which will be subsequently discussed.

It is informative to know the reliability (or uncertainty) of each of the biological design parameters computed by the program (e.g., Table A-1). The theory of propagation of errors must be used to develop this information. These design parameters are computed by various mathematical manipulations involving addition, subtraction, multiplication, and division. Since the statistical information for each of the components entering into the calculation of these design parameters was previously calculated, it is possible to apply appropriate techniques to estimate the uncertainty in the latter. A measure of this uncertainty is the probable uncertainty or error. This statistic assumes that the directly measured quantities (input parameters) will differ from their true values by amounts less than their maximum uncertainties (as represented by the standard deviation of the mean), and that some of the measured values will be larger than their true values while others will be smaller. The probable uncertainty is computed with the equation:

$$\sigma_y = \left(\frac{\partial y}{\partial x_1}\right)^2 \sigma_{x_1}^2 + \left(\frac{\partial y}{\partial x_2}\right)^2 \sigma_{x_2}^2 + \ldots + \left(\frac{\partial y}{\partial x_n}\right)^2 \sigma_{x_n}^2$$

TABLE A-1

PARAMETERS USED IN PROCESS DESIGN

Influent Parametric Relationships	Removal Efficiencies $(\frac{S_o - S_e}{S_o} \times 100)$	Design Parameters	Removal Velocities $\frac{(S_o - S_e)}{X_a t}$	Organic Loading $\frac{S_o}{X_a t}$
Total: BOD_5/COD	Total BOD_5	Sludge Age	Soluble BOD_5	Soluble BOD_5
Soluble: BOD_5/COD	Settled BOD_5	Respiration - R_r/X_a	Soluble COD	Soluble COD
Total: BOD_5/TOC	Soluble BOD_5	$X_a \cdot t$	Soluble TOD	Soluble TOD
Soluble: BOD_5/TOC	Total COD	ΔX_a	Soluble TOC	Soluble TOC
Total:. BOD_5/TOD	Settled COD	$\Delta X_a/X_a$		
Soluble: BOD_5/TOD	Soluble COD	Detention Time		
Total: COD/TOC	Total TOD			
Soluble: COD/TOC	Settled TOD			
	Soluble TOD			
	Total TOC			
	Settled TOC			
	Soluble TOC			

where:

$\sigma_{\bar{y}}$ = probable uncertainty of design parameter

y = design parameter $f(x_1, x_2, \ldots, x_n)$

$x_1, x_2, \ldots, x_n$ = directly measured input parameters

$\sigma_{\bar{x}_1}, \sigma_{\bar{x}_2}, \ldots, \sigma_{\bar{x}_n}$ = standard deviation of the mean for input parameters

The above equation indicates that the error in the computed design parameter is not directly influenced by the nature of the equation used to calculate it, but rather is a function of the errors in the independent variables as modified or propagated by the equation. Any input parameter with a standard deviation of the mean of zero does not contribute to the error in the design parameter in any way. Thus, the uncertainty in the design parameter is a function of the "weakest link" or "links" as the case may be.

After the probable uncertainty is computed for each design parameter in each of the data sets, all of this information is printed out in a readily usable format. Selected design parameters are used later in the program computational scheme to compute design coefficients which are useful in the development of a biological waste treatment process design.

Design Coefficients

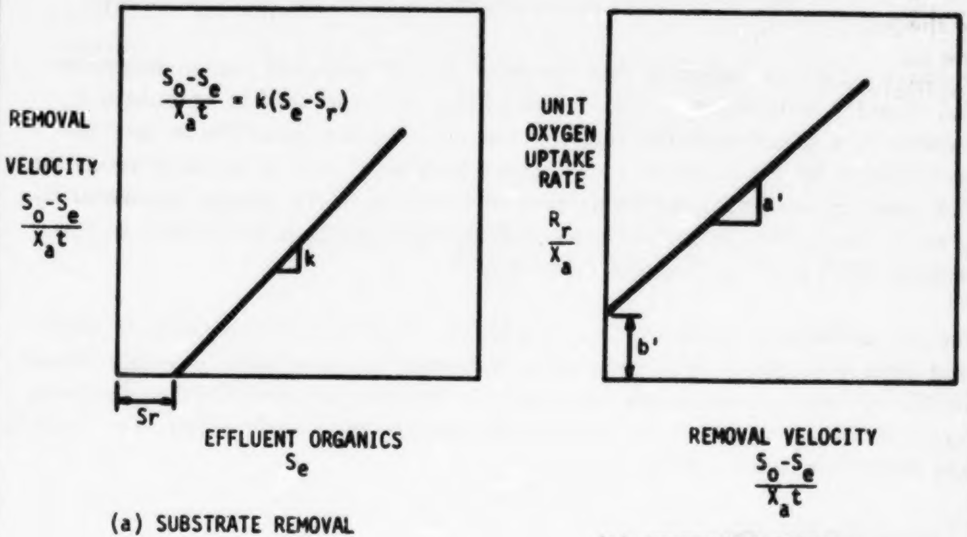
Certain biological waste treatment design coefficients useful in computing sludge growth rates, process oxygen requirements, and organic removal rates must be calculated by fitting linear relationships to design parameters measured at several organic loadings (food to micro-organism ratios). Program STATPK provides the user with the option to compute these coefficients directly if the appropriate data are available. Figure A-2 illustrates the relationships developed in this program and the design coefficients which are calculated from these relationships. These coefficients are computed for each of the types of organic parameters in the original data set (e.g., BOD_5, COD, TOC, TOD), as applicable.

It is inadvisable to use this curve-fitting technique if only two sets of data are available, and obviously it is meaningless for data from only one loading. As mentioned above, a linear relationship is assumed for design coefficient calculation which is consistent with their theoretical development.

The technique used for estimating the regression coefficients of the assumed linear relationship is the method of least squares. The appropriate intercepts and slopes which represent the biological design coefficients are the results of these regression analyses.

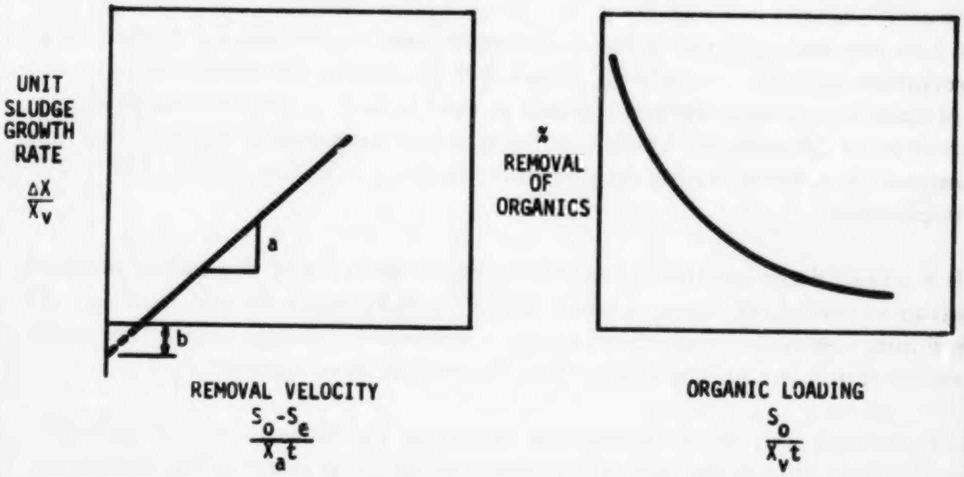
Figure A-2

RELATIONSHIPS FOR DETERMINING DESIGN COEFFICIENTS



(a) SUBSTRATE REMOVAL

(b) OXYGEN UTILIZATION



(c) SLUDGE PRODUCTION

(d) ORGANIC REMOVAL EFFICIENCY

Several statistical parameters are computed to indicate the goodness of fit or reliability of each regression line. This information includes the sum of the squares, the correlation coefficient and the index of correlation. The sum of the squares is obtained directly from the least squares analysis and represents the minimized residuals between the measured parameter values and the fitted line. The correlation coefficient is simply a measure of correlation between the two variables being analyzed and is not a measure of the goodness of fit of the regression line. However, it is useful in determining the confidence in the design coefficients obtained from the regression analysis. The correlation coefficient is calculated from the equation:

$$r = \frac{\sum_{i=1}^{n} (x_i - \bar{x})(y_i - \bar{y})}{n S_x S_y} \qquad \text{(A-9)}$$

where:

r = correlation coefficient
x_i = ith value of design parameter used as independent variable
y_i = ith value of design parameter used as dependent variable
$\bar{x}, \bar{y}$ = means
S_x, S_y = standard deviations
n = number of observations

Correlation coefficients are limited to the range:

$$1.0 > r > -1.0$$

Negative correlation coefficients denote an inverse relationship between the variables. Coefficients with an absolute value of 0.9 or greater demonstrate a strong relationship between variables and would indicate that curve fitting should be quite successful. Conversely, correlation coefficients with absolute values less than 0.7 indicate that the relationship between the variables is very weak and that curve fitting would likely be unsuccessful.

The index of correlation is a measure of the accuracy of fit of an equation to a set of experimental data. This statistic is a function of the standard deviation of the data with respect to the fitted curve and of the apparent standard deviation of the data with respect to their mean value. The index of correlation is computed as:

$$I = \sqrt{1 - \frac{\sum_{i=1}^{n} (y_i - y_c)^2 \,/\, (n-2)}{\sum_{i=1}^{n} (y_i - \bar{y})^2 \,/\, (n-1)}} \qquad \text{(A-10)}$$

where:

I = index of correlation
y_i = measured ith value of dependent variable @ $x = x_i$
y_c = computed value of dependent variable @ $x = x_i$
$\bar{y}$ = mean of dependent variable
n = number of observations

Values of I range from 0 to 1.0. A regression equation is considered to fit the measured data well if the index of correlation lies between 0.94 and 1.0. Lesser values of this index indicate a poorer fit and thus reduce the reliability of the design coefficients obtained from the regression analysis.

These measures of goodness of fit are computed for each of the relationships shown in Figure A-2. Applying these criteria with engineering judgment permits an evaluation of the reliability of the biological design coefficients. In addition, the use of the method of least squares to fit the linear relationships assures that the most reliable fit of the experimental data has been obtained, regardless of the degree of correlation of the data.

Information Display

All of the input data, the design parameters and their associated statistics are printed in a readily usable tabular format. This listing is designed so that it can fit into a standard three-ring notebook. The linear relationships used to calculate the biological waste treatment process design coefficients are plotted graphically. The design coefficients themselves and the goodness-of-fit measures are printed out in a tabular format. The program user has an option in regard to the type of graphical display he uses to plot the linear relationships.

One option is to use a drum-type pen plotter to graph each of the relationships. This plotter is found as a peripheral unit of many high-speed computers. The plot routine in this program is machine-dependent and will operate only on the Univac 1108 Computer. The pen plots from this option was suitable for direct inclusion in engineering reports. They include titles, labeled axes, the linear relationships and the measured data points. A complete graphical plot of the type shown in Figure A-2 can be completed in approximately seven minutes with the Univac plotting system.

The other plotting option is executed on the standard line printer and is commonly known as a "printer plot." Standard printer characters are used to generate the plot which includes labeled axes, titles, the regression line and the measured data for each relationship. The relationship can be traced directly from the printer plot to standard paper and can be used in a report with appropriate labeling lettered in. Each of the design relationships shown in Figure A-2 may be plotted in this manner.

Caveat

Program STATPK is a powerful tool for analyzing experimental biological waste treatment process data and as such it should be used with care and judgment. The design parameters and coefficients calculated by this program are no better than the input data and should be considered in this context.

Particular care should be taken in the interpretation of the statistical analyses. As was previously discussed, most of these analyses assume that the data are taken from a normal (Gaussian) distribution. Slight deviations from this distribution type will cause no problems, but serious discrepancies can arise if unusual population distributions exist. An example of this is an industrial waste which is subject to large dumps and spills and which is in fact a combination of several different populations. The normality assumption will fail completely in this case and the computed statistical measures will be meaningless. The user of the program should be aware of the characteristics of the waste and process being analyzed so that he can find any ambiguties in the input data. If a discrepancy in the statistical analysis is suspected, it is wise to graphically display the probability distribution of the suspect parameter on probability graph paper so that it may be closely examined to verify or reject the normality assumption.

The other assumptions made in the statistical analyses, such as the assumption of equality of variances for the t-Test, should also be considered when using the results of the program. In conclusion, this program uses the best data analysis techniques available for its purpose, but none are universally applicable. In the final analysis, only sound engineering judgment can provide the desired confidence in the final design.

Summary

This section has included a brief description of the STATPK program which was used to resolve the pilot plant data into the necessary design parameters, coefficients, and constants with the corresponding statistical accuracy. Although the Program was developed specifically for the Deepwater Pilot Plant Study because of the myriad of data accumulated, it will have application for similar projects requiring biological process kinetics and coefficient derivation.

REFERENCES -- APPENDIX A

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